THE NWS DAMBRK MODEL:

THEORETICAL BACKGROUND/USER DOCUMENTATION

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by

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TABLE OF CONTENTS

APPENDICES:

APPENDICES (CONTINUED):

THE NWS DAMBRK MODEL:
THEORETICAL BACKGROUND/USER DOCUMENTATION

Abstract

A dam-break flood forecasting model (DAMBRK) is described and applied to two actual dam-break flood waves. The model consists of a breach component which utilizes simple parameters to provide a temporal and geometrical description of the breach. The model computes the reservoir outflow hydrograph resulting from the breach via a broad-crested weir flow approximation, which includes effects of submergence from downstream tailwater depths and corrections for approach velocities. Also, the effects of storage depletion and upstream inflows on the computed outflow hydrograph are accounted for through storage routing within the reservoir. The basic component of the DAMBRK model ia a dynamic routing technique for determining the modifications to the dam-break flood wave as it advances through the downstream valley, including its travel time and resulting water surface elevations. The dynamic routing component is based on a weighted four-point, nonlinear finite-difference solution of the one-dimensional equations of unsteady flow (Saint-Venant equations) which allows variable time and distance steps to be used in the solution procedure. Provisions are included for routing supercritical flows, subcritical flows, or a spontaneous mixture of each, and incorporating the effects of downstream obstructions such as road-bridge embankments and/or other dams, routing mud/debris flows, pressurized flow, landslide-generated reservoir waves, accounting for volume and flow losses during the routing of the dam-break wave, considering the effects of off-channel (dead storage), floodplains, and floodplain compartments. Model input/output may be in either English or metric units. Modeling difficulties and parameter uncertainties are described and methods of treating them are discussed. Model data requirements are flexible, allowing minimal data input when it is not available while permitting extensive data to be used when appropriate. The model was tested on the Teton Dam failure and the Buffalo Creek coal-waste dam collapse. Computed outflow volumes through the breaches coincided with the observed values in magnitude and timing. Observed peak discharges along the downstream valleys were satisfactorily reproduced by the model even though the flood waves were severely attenuated as they advanced downstream. The computed peak flood elevations were within an average of 1.9 ft and 2.1 ft of the observed maximum elevations for Teton and Buffalo Creek, respectively. Both the Teton and Buffalo Creek simulations indicated an important lack of sensitivity of downstream discharge to errors in the forecast of the breach size and timing. Such errors produced significant differences in the peak discharge in the vicinity of the dams; however, the differences were rapidly reduced as the waves advanced downstream. Computational requirements of the model are quite feasible for mainframe, mini- or microcomputers. Suggested ways for using the DAMBRK model in preparation of pre-computed flood information and in real-time forecasting are presented along with several examples illustrating the use of the DAMBRK model.

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1. INTRODUCTION

Dams provide society with essential benefits such as water supply, flood control, recreation, hydropower, and irrigation. However, catastrophic flooding occurs when a dam fails and the impounded water escapes through the breach into the downstream valley. Usually, the magnitude of the flow greatly exceeds all previous floods and the response time available for warning is much shorter than for precipitation-runoff floods. According to reports by the International Commission on Large Dams (ICOLD, 1973) and the United States Committee on Large Dams in cooperation with the American Society of Civil Engineers (ASCE/USCOLD, 1975), about 38% of all dam failures are caused by overtopping of the dam due to inadequate spillway capacity and by spillways being washed out during large inflows to the reservoir from heavy precipitation runoff. About 33% of dam failures are caused by seepage or piping through the dam or along internal conduits, while about 23% of the failures are associated with foundation problems, and the remaining failures are due to slope embankment slides, damage or liquefaction of earthen dams from earthquakes, and landslide-generated waves within the reservoir. Middlebrooks (1952) describes earthen dam failures that occurred within the U.S. prior to 1951. Johnson and Illes (1976) summarize 300 dam failures throughout the world.

The potential for catastrophic flooding due to a dam failure was brought to the Nation's attention during the 1970's by several floods due to dam failures such as the Buffalo Creek coal-waste dam, the Teton Dam, the Toccoa Dam, and the Laurel Run Dam. Also, there are many dams that are 30 or more years old, and many of the older dams are a matter of serious concern because of increased hazard potential due to downstream development and increased risk of

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failure due to structural deterioration or inadequate spillway capacity. A report by the U.S. Army (1981) gives an inventory of the Nation's approximately 70,000 dams with heights greater than 25 ft or storage volumes in excess of 50 acre-ft. The report also classifies some 20,000 of these as being "so located that failure of the dam could result in loss of human life and appreciable property damage..."

The National Weather Service (NWS) has the responsibility to advise the public of downstream flooding when there is a failure of a dam. Although this type of food has many similarities to floods produced by precipitation runoff, the dam-break flood has some very important differences which make it difficult to analyze with the common techniques which have worked so well for the precipitation-runoff floods. To aid NWS flash flood hydrologists who are called upon to forecast the downstream flooding (flood inundation information and warning times) resulting from dam-failures, a numerical model (DAMBRK) has been developed. The DAMBRK model may also be used for a multitude of purposes by engineering planners, designers, and analysts who are concerned with possible future flood inundation mapping due to dam-break floods and/or reservoir spillway floods. The DAMBRK model can also be used for routing any specified flood hydrograph through reservoirs, rivers, canals, or estuaries as part of general engineering studies of waterways. Its principal limitation is its confinement to analyzing flow through a single waterway rather than a network of mutually interactive channels, e.g., dendritic (tree-type network of rivers, distributary network of irrigation canals, and estuarial network of waterways. Two other NWS models may be used for channel networks, DWOPER (Fread, 1978, 1983), and FLDWAV (Fread, 1985b; Fread and Lewis, 1988). The models are available for mainframe, mini- ,or microcomputers. The FLDWAV model is scheduled for release sometime during the latter part of 1988.

1.1 Model Development

The DAMBRK model represents the current state-of-the-art in understanding of dam failures and the utilization of hydrodynamic theory to predict the dam-break wave formation and downstream progression. The model has wide applicability; it can function with various levels of input data ranging from rough estimates to complete data specification; the required data is readily accessible; and it is economically feasible to use, i.e., it requires minimal computational effort on mainframe computing facilities and is feasible for use on microcomputers (IBM PC compatible). DAMBRK is used by most federal/state agencies in the U.S. and in over forty nations around the world. It is also extensively used by private consultants, hydro-power and mining companies, and is utilized in more than 40 universities for teaching and research purposes.

The model consists of two conceptual parts, namely: (1) description of the dam failure mode, i.e., the temporal and geometrical description of the breach; and (2) a hydraulic computational algorithm for determining the time history (hydrograph) of the outflow through the breach as affected by the breach description, reservoir inflow, reservoir storage characteristics, spillway outflows, and downstream tailwater elevations; and for routing of the outflow hydrograph through the downstream valley in order to account for the changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams. The model also determines the resulting water surface elvations (stages) and flood-wave travel times.

The latest version of DAMBRK is an expanded version of a practical operational model first presented in 1977 by the author (Fread, 1977). Other versions were previously released in 1979, 1980, 1981, 1982, and 1984 as reported by the author in a paper titled "The NWS Dam-Break Flood Forecasting Model" (Fread, 1984b). The first model was based on previous work by the author on modeling breached dams (Fread and Harbaugh, 1973) and routing of flood waves (Fread, 1974a, 1976). There have been a number of other operational dam-break models that have appeared in the literature, e.g., Price, et al. (1977), Gundlach and Thomas (1977), Thomas (1977), Keefer and Simons (1977), Chen and Druffel (1977), Balloffet, et al. (1974), Balloffet (1977), Brown and Rogers (1977), Rajar (1978), Brevard and Theurer (1979), Bodine, and HEC (1981). DAMBRK differs from each of these models in either or all of the following essential functions: the treatment of the breach formation, the outflow hydrograph generation, and the downstream flood routing.

During the last 6-7 years, there have been a number of studies in which various models suitable for dam-break analysis were compared, e.g., Land (1980) with the U.S. Geological Survey; McMahon (1981) with the U.S. Corps of Engineers; Tschantz and Mojib (1981) with the University of Tennessee; Singh and Snorrason (1982) with the Illinois State Water Survey; Keefer and Peck (1982), and Binnie & Partners (1986) from the private consulting sphere; and Wurbs (1985, 1986) with Texas A & M University. The general conclusion of these studies was that DAMBRK is preferred over the other models on the basis of accuracy, theoretical foundation, range of applicability, and relative ease of application. However, the DAMBRK model's limitations and difficulties in application were pointed out in these studies. Research has been on-going in developing improvements in the DAMBRK model allowing it to have fewer limitations, and an increasing range of applicability and numerical robustness for more convenient usage.

1.2 Scope

Herein, the 1988 version of DAMBRK is described. New developments are delineated, information on model application difficulties along with suggested means of overcoming the difficulties are provided, some example applications are given, a data input description along with some examples are provided, and model output is described.

1.3 Summary Preview of DAMBRK

DAMBRK is used to develop the outflow hydrograph from a dam and hydraulically route the flood through the downstream valley. The governing equations of the model are the complete one-dimensional Saint-Venant equations of unsteady flow which are coupled with internal boundary equations representing the rapidly varied (broad-crested weir) flow through structures such as dams and bridge/embankments which may develop a time-dependent breach. Also, appropriate external boundary equations at the upstream and downstream ends of the routing reach are utilized. The system of equations is solved by a nonlinear weighted 4-point implicit finite-difference method. The flow may be either subcritical or supercritical or a combination of each varying in space and time from one to the other; fluid properties may obey either the principles of Newtonian (water) flow or non-Newtonian (mud/debris flows or the contents of a mine-tailings dam) flow. The hydrograph to be routed may be specified as an input time series or it can be developed by the model using specified breach parameters (size, shape, time of development). The possible presence of downstream dams which may be breached by the flood, bridge/embankment flow constrictions, tributary inflows, river sinuosity, levees located along the downstream river, and tidal effects are each properly considered during the downstream propagation of the flood. DAMBRK also may be used to route mud and debris flows or rainfall/snowmelt floods using specified upstream hydrographs. High water profiles along the valley, flood arrival times, and hydrographs at user selected locations are standard model output. Model input/output may be in either English or metric units.

2. BREACH DESCRIPTION

The breach is the opening formed in the dam as it fails. The actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously. Investigators of dam-break flood waves such as Ritter (1892), Schocklitsch (1917), Re (1946), Dressler (1954), Stoker (1957), Su and Barnes (1969), and Sakkas and Strelkoff (1973) assumed the breach encompasses the entire dam and that it occurs instantaneously. Others, such as Schocklitsch (1917) and Army Corps of Engineers (1960), have recognized the need to assume partial rather than complete breaches; however, they assumed the breach occurred instantaneously. The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. These assumptions are somewhat appropriate for concrete arch dams, but they are not appropriate for earthen dams and concrete gravity dams. In DAMBRK the breach is always assumed to develop over a finite interval of time (τ) and will have a final size determined by a terminal bottom width parameter (b) and various shapes depending on another parameter (Z) as shown in Fig. 1. Such a parametric representation of the breach is utilized in DAMBRK for reasons of simplicity, generality, wide applicability, and the uncertainty in the actual failure mechanism. This approach to the breach description follows that used by Fread and Harbaugh (1973).

The shape parameter (Z) identifies the side slope of the breach, i.e., 1 vertical: Z horizontal. The range of Z values is from 0 to somewhat larger than unity. Its value depends on the angle of repose of the compacted and wetted materials through which the breach develops. Rectangular, triangular, or trapezoidal shapes may be specified by using various combinations of values for Z and b, e.g., Z=0 and b>0 produces a rectangle and Z>0 and b=0 yields a triangular-shaped breach. The terminal width b is related to the average width of the breach ($\bar{b}$) by the following:

$$b = \bar{b} - Zh_d \quad \text{.....(1)}$$

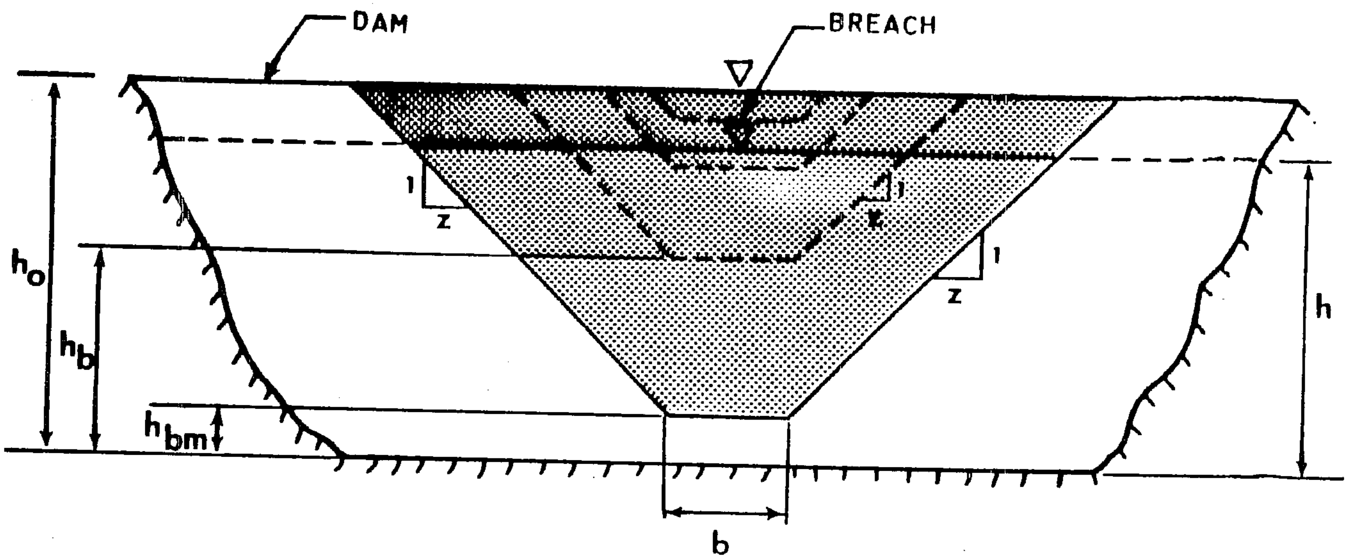


Fig. 1 — FRONT VIEW OF DAM SHOWING FORMATION OF BREACH

The model assumes the breach bottom width starts at a point (see Fig. 1) and enlarges at a linear or nonlinear rate over the failure time (τ) until the terminal bottom width (b) is attained and the breach bottom has eroded to the elevation h_{bm}. If τ is less than one minute, the width of the breach bottom starts at a value of b rather than zero. This represents more of a collapse failure than an erosion failure. The bottom elevation of the breach is simulated as a function of time (τ) according to the following:

$$h_b = h_d - (h_d - h_{bm})\left(\frac{t_b}{\tau}\right)^{\rho} \qquad \text{if} \quad 0 < t_b \leq \tau \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which h_{bm} is the final elevation of the breach bottom which is usually, but not necessarily, the bottom of the reservoir or outlet channel bottom, t_b is the time since beginning of breach formation, and ρ is the parameter specifing the degree of nonlinearity, e.g., $\rho=1$ is a linear formation rate, while $\rho=2$ is a nonlinear quadratic rate; the range for ρ is $1 \leq \rho \leq 4$; however, the linear rate is usually assumed. The instantaneous bottom width (b_i) of the breach is given by the following:

$$b_i = b(t_b/\tau)^{\rho} \qquad \text{if} \quad 0 < t_b \leq \tau \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

During the simulation of a dam failure, the actual breach formation commences when the reservoir water surface elevation (h) exceeds a specified value, h_f. This feature permits the simulation of an overtopping of a dam in which the breach does not form until a sufficient amount of water is flowing over the crest of the dam. A piping failure may also be simulated by specifing the initial centerline elevation of the pipe. A new feature in DAMBRK allows the user to specify the time after beginning of simulation when the breach begins to form. This is an alternative to the use of h_f as the overtopping elevation at which failure commences. Also, the use of ρ in Eqs. (2-3) with $\rho>1$ is a new feature in DAMBRK. Another new feature, is the ability to limit the breach formation to the spillway section of the dam.

2.1 Concrete Dams

Concrete gravity dams tend to have a partial breach as one or more monolith sections formed during the construction of the dam are forced apart and over-turned by the escaping water. The time for breach formation is in the range of a few minutes. It is difficult to predict the number of monoliths which may be displaced or fail; however, by using the DAMBRK model and making several separate applications wherein the parameter b representing the combined lengths of assumed failed monoliths is varied in each, the resulting reservoir water surface elevations and breach velocities can be used to indicate the extent of reduction of the loading pressures on the dam. Since the loading diminishes as b is assumed to increase, a limiting safe loading condition which would not cause further failure may be estimated. Concrete arch dams tend to fail completely and are assumed to require only a few minutes for the breach formation. The shape parameter (Z) is usually assumed zero for concrete dams.

2.2 Earthen Dams

Earthen dams which exceedingly outnumber all other types of dams do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width ($\bar{b}$) in the range ($h \le \bar{b} \le 5h_d$) where h_d is the height of the dam. The middle portion of this range for $\bar{b}$ is supported by the summary report of Johnson and Illes (1976) and the upper range by the report of Singh and Snorrason (1982). Breach widths for earthen dams are therefore usually much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time (τ) for its formation through erosion of the dam materials by the escaping water. Total time of failure (for overtopping) may be in the range of a few minutes to usually less than an hour, depending on the height of the dam, the type of materials used in construction, the extent of compaction of the materials, and the magnitude and duration of the overtopping flow of the escaping water. The time of failure as used in DAMBRK is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face. Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by the escaping water. Times of failure are usually considerably longer for piping than overtopping failures since the upstream face is slowly being eroded in the very early phase of the piping development. As the erosion proceeds, a larger and larger opening is formed; this is eventually hastened by caving-in of the top portion of the dam. Poorly constructed coal-waste slag piles (dams) which impound water tend to fail within a few minutes, and have average breach widths in the upper range of the earthen dams mentioned above.

Recently some statistically derived predictors for $\bar{b}$ and τ have been presented in the literature, i.e., MacDonald and Langridge-Monopolis (1984) and Froelich (1987). From Froelich's work in which he used the properties of 43 breaches of dams ranging in height from 15 to 285 ft with all but 6 between 15 and 100 ft, the following predictive equations can be obtained:

$$\bar{b} = 9.5\, k_o\, (V_r\, h_d)^{0.25} \quad \text{...(4)}$$

$$\tau = 0.59\, V_r^{0.47} \;/\; h_d^{0.9} \quad \text{...(5)}$$

in which $\bar{b}$ is average breach width (ft), τ is time of failure (hrs), $k_o = 0.7$ for piping and 1.0 for overtopping, V_r is volume (acre-ft) and h_d is the height (ft) of water over the breach bottom which is usually about the height of the dam. Standard error of estimate for $\bar{b}$ was ±94 ft which is an average error of ±54% of $\bar{b}$, and the standard error of estimate for τ was ±0.9 hrs which is an average error of ±70% of τ.

Another means of determining the breach properties is the use of physically based breach erosion models. Cristofano (1965) attempted to model the partial, time-dependent breach formation in earthen dams; however, this procedure requires critical assumptions and specification of unknown critical parameter values. Also, Harris and Wagner (1967) used a sediment transport relation to determine the time for breach formation, but this procedure requires specification of breach size and shape in addition to two critical parameters for the sediment transport relation. More recently, Ponce and Tsivoglou (1981) presented a rather computationally complex breach erosion model which coupled the Meyer-Peter and Muller sediment transport equation to the one-dimensional differential equations of unsteady flow and sediment conservation. They compared the model's predictions with observations of a breached landslide-formed dam on the Mantaro River in Peru. The results were substantially affected by the judicious selection of the Manning n, a breach width-flow relation parameter, and a coefficient in the sediment transport equation.

Recently the author (Fread, 1984a, 1987a) developed a breach erosion model for earthen dams. It substantially differs from the previously mentioned models. It is a physically based mathematical model which predicts the breach characteristics (size, shape, time of formation) and the discharge hydrograph emanating from a breached earthen dam. The earthen dam may be man-made or naturally formed by a landslide. The model is developed by coupling the conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity of the unsteady uniform flow along an erosion-formed breach channel. The bottom slope of the breach channel is assumed to be essentially that of the downstream face of the dam. The growth of the breach channel is dependent on the dam's material properties (D_{50} size, unit weight, friction angle, cohesive strength). The model considers the possible existence of the following complexities: (1) core material having properties which differ from those of the outer portions of the dam; (2) the necessity of forming an eroded ditch along the downstream face of the dam prior to the actual breach formation by the overtopping water; (3) the downstream face of the dam can have a grass cover or be composed of a material of larger grain size than the outer portion of the dam; (4) enlargement of the breach through the mechanism of one or more sudden structural collapses of portions of the dam where breaching occurs due to the hydrostatic pressure force exceeding the resisting shear and cohesive forces; (5) enlargement of the breach width by collapse of breach sides according to slope stability theory; and 6) initiation of the breach via piping with subsequent progression to a free surface breach flow. The outflow hydrograph is obtained through a time-stepping iterative solution that requires only a few seconds for computation on a main-frame computer. The model is not subject to numerical stability or convergence difficulties. The model's predictions have been favorably compared with observations of a piping failure of the man-made Teton Dam in Idaho, the piping failure of the man-made Lawn Lake Dam in Colorado, and an overtopping activated breach of a landslide-formed dam in Peru. Model sensitivity to numerical parameters is minimal; however, it is sensitive to the internal friction angle of the dam's material and the extent of grass cover when simulating man-made dams and to the cohesive strength of the material composing landslide-formed dams. In the three test cases, a reasonable variation of cohesion and internal friction angle produced less than ±20% variation in the breach properties. The BREACH model has not been directly incorporated into DAMBRK to discourage its indiscriminate use, since it should be used judiciously and with caution until it receives further verification. BREACH is intended to be an auxillary method for determining the breach parameters and to be used in conjunction with statistical and range of magnitude data from historical breaches.

Another way of checking the reasonableness of the breach parameters ($\bar{b}$ and τ) is to use the following the equations:

$$Q_p^* = 370\ (V_r\ h_d)^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$Q_p = 3.1\ \bar{b}\ \left(\frac{C}{\tau + C/\sqrt{h_d}}\right)^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which Q_p* and Q_p are the expected peak discharge (cfs) through the breach, V_r and h_d are the reservoir volume (acre-ft) and height (ft) of dam, respectively and $C = 23.4\ A_s/\bar{b}$ in which A_s is the surface area (acres) of the reservoir at the top of the dam. Eq. (6) was developed by Hagen (1982) for historical data from 14 dam failures and provides a maximum envelope of all 14 of the observed discharges. Eq. (7) was developed by the author (1981) and is used in the NWS Simplified Dam Break Model, SMPDBK (Wetmore and Fread, 1984). After selecting $\bar{b}$ and τ, Eq. (7) can be used to compute Q_p which then can be compared with Q_p^* from Eq. (6). Thus, if $Q_p >> Q_p^*$, then either $\bar{b}$ is too large and/or τ is too small; however if $Q_p << Q_p^*$ then either $\bar{b}$ is too small and/or τ is too large. The author has found that Eq. (6) over-estimated the peak discharges for each of 21 dam failures (including the previously mentioned 14 failures) by an average of 130 percent. Eq. (7), although not used in DAMBRK, has been found to yield peak discharges within 5-10 percent of those produced in DAMBRK when equivalent values of $\bar{b}$ and τ are utilized in Eq. (7) and in Eqs. (2-3) within DAMBRK.

2.3 Parameter Sensitivity

Selection of breach parameters before a breach forms, or in the absence of observations, introduces a varying degree of uncertainty in the downstream flooding results of the DAMBRK model; however, errors in the breach description and thence in the resulting peak outflow rate are damped-out as the flood wave advances downstream. Using DAMBRK, it has been observed that varations in Q_p at the dam are damped-out as the flood peak advances farther and farther downstream. The extent of damping is related to the size of the downstream floodplain; the wider the floodplain, the greater will be the extent of damping. Sensitivity tests on the breach parameters are best determined using the DAMBRK model and then comparing the variation in simulated flood peaks at critical downstream locations. In this way, the real uncertainty in the breach parameter selections will be determined.

For conservative forecasts which err on the side of larger flood waves, values for b and Z should produce an average breach width ($\bar{b}$) in the uppermost range for a certain type of dam. Failure time (τ) should be selected in the lower range to produce a maximum outflow. Of course, observational estimates of $\bar{b}$ and τ should be used when available to update forecasts when response time is sufficient as in the case of forecast points many miles downstream of the breached dam. Flood wave travel rates are often in the range of 2-10 miles per hour. Accordingly, response times for some downstream forecast points may therefore be sufficient for updated forecasts to be issued.

Also, Eq. (7) can be used quickly and conveniently to test the sensitivity of $\bar{b}$ and τ for a specific reservoir having properties of V_r, h_d, and A_s. For example, using Eq. (7) for a moderatley large reservoir (V_r = 250,000 acre-ft, h_d = 260 ft, A_s = 2,000 acres), it can be shown that Q_p varies in proportion as $\bar{b}$ varies, however, Q_p only varies by less than 1/5 of the variation in τ. Although for a fairly small reservoir (V_r = 500 acre-ft, h_d = 40 ft, A_s = 10 acres), it can be shown, using Eq. (7), that Q_p varies less than 20 percent for a variation in $\bar{b}$ of 50 percent while Q_p varies about 40 percent for a variation in τ of 50 percent. Thus, it might be generalized, that, for large reservoirs Q_p is quite sensitive to $\bar{b}$ and rather insensitive to τ, while for very small reservoirs Q_p is somewhat insensitive to $\bar{b}$ and fairly sensitive to τ.

3. HYDRAULIC COMPUTATIONAL ALGORITHM

The essential component of the DAMBRK model is the hydraulic computational algorithm. It is used to compute the outflow from a breached dam in conjunction with (1) a parametric description of the breach size and shape which varies with time and (2) a description of spillway characteristics. The hydraulic computational algorithm also determines the extent and time of occurrence of flooding in the downstream valley as determined by routing the outflow hydrograph through the valley. The hydrograph is modified (attenuated, lagged, and distorted) as it is routed through the valley due to the effects of valley storage, frictional resistance to flow, floodwave acceleration components, flow losses, and downstream channel constrictions and/or flow control structures. Modifications to the dam-break flood wave are manifested as attenuation of the flood peak magnitude, spreading-out or dispersion of the temporal varying flood-wave volume, and changes in the celerity (propagation speed) or travel time of the flood wave. If the downstream valley contains significant storage volume such as a wide floodplain, the flood wave can be extensively attenuated and its time of travel greatly increased. Even when the downstream valley approaches that of a uniform rectangular-shaped section, there is appreciable attenuation of the flood peak and reduction in the wave celerity as the wave progresses through the valley.

A distinguishing feature of dam-break waves is the great magnitude of the peak discharge when compared to runoff-generated flood waves having occurred in the past in the same valley. The dam-break flood is usually many times greater than the runoff flood of record. The above-record discharges make it necessary to extrapolate certain coefficients used in various flood routing techniques and make it impossible to fully calibrate the routing technique.

Another distinguishing characteristic of dam-break floods is the very short duration time, and particularly the extremely short time from beginning of rise until the occurrence of the peak. The time to peak, in almost all instances, is synonymous with the breach formation time (τ) and therefore is in the range of a few minutes to a few hours. This feature along with the great magnitude of the peak discharge causes the dam-break flood wave to have acceleration components of a far greater significance than those associated with a runoff-generated flood wave.

There are two basic types of flood routing methods, the hydrologic and the hydraulic methods. (See Fread (1985b) for a more complete description of the two types of routing methods.) The hydrologic methods usually provide a more approximate analysis of the progression of a flood wave through a river reach than do the hydraulic methods. The hydrologic methods are used for reasons of convenience and economy. They are most appropriate, as far as accuracy is concerned, when the flood wave is not rapidly varying, i.e., the flood-wave acceleration effects are negligible compared to the effects of gravity and channel friction. Also, they are best used when the flood wave is very similar in shape and magnitude to previous flood waves for which stage and discharge observations are available for calibrating the hydrologic routing parameters (coefficients).

For routing dam-break flood waves, a particular hydraulic method known as the dynamic wave method is chosen. This choice is based on its ability to provide more accuracy in simulating the dam-break flood wave than that provided by the hydrologic methods, as well as, other less complex hydraulic methods such as the kinematic wave and the diffusion wave methods. Of the many available hydrologic and hydraulic routing techniques, only the dynamic wave method accounts for the acceleration effects associated with the dam-break wave and the influence of downstream unsteady backwater effects produced by channel constrictions, dams, bridge-road embankments, and tributary inflows. Also, the dynamic wave method can be used economically, i.e., the computational costs can be made rather insignificant if advantages of certain "implicit" numerical solution techniques are utilized. Also, the current use of microcomputers has reduced the significance of computational costs.

The dynamic wave method is based on the complete one-dimensional equations of unsteady flow which are used to route the dam-break flood hydrograph through the downstream valley. This method is based on an expanded version of the original equations developed by Barre De Saint-Venant (1871). The only coefficient that must be extrapolated beyond the range of past experience is the coefficient of flow resistance. It so happens that this is usually not an extremely sensitive parameter in effecting the modifications of the flood wave due to its progression through the downstream valley. The applicability of

Saint-Venant equations to simulate abrupt waves such as the dam-break wave has been demonstrated by Terzidis and Strelkoff (1970) and by Martin and Zovne (1971) who used a "through computation" method which does not provide special treatment for shock waves. DAMBRK uses the "through computation" method. The DAMBRK model does not isolate a single shock wave, should it occur, nor apply the shock equations to it while using the Saint-Venant equations for all other portions of the flow.

3.1 Expanded Saint-Venant Equations

The equations of Saint-Venant, expressed in conservation form (Fread, 1974b), with additional terms for the effect of expansion/contractions (Fread, 1976), channel sinuosity (DeLong, 1986) and non-Newtonian flow (Fread, 1987b) consist of a conservation of mass equation, i.e.,

$$\frac{\partial Q}{\partial x} + \frac{\partial s_c(A+A_o)}{\partial t} - q = 0 \quad \text{(8)}$$

and a conservation of momentum equation, i.e.

$$\frac{\partial(s_m Q)}{\partial t} + \frac{\partial(\beta Q^2/A)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e + S_i\right) + L' = 0 \quad \text{(9)}$$

where h is the water surface elevation, A is the active cross-sectional area of flow, A_o is the inactive (off-channel storage) cross-sectional area, s_c and s_m are sinuosity factors after DeLong (1986) which vary with h, x is the longitudinal distance along the channel (valley), t is the time, q is the lateral inflow or outflow per lineal distance along the channel (inflow is positive and outflow is negative in sign), β is the momentum coefficient for velocity distribution, g is the acceleration due to gravity, S_f is the boundary friction slope, S_e is the expansion-contraction slope, and S_i is the additional friction slope associated with internal viscous dissipation of non-Newtonian fluids such as mud/debris flows.

In Eq. (9), L' is the momentum effect of lateral flow assumed herein to enter or exit perpendicular to the direction of the main flow. This term (Strelkoff, 1969) has the following form: (1) lateral inflow, L' = 0; (2) seepage lateral outflow, L' = -0.5qQ/A; and (3) bulk lateral outflow, L' = -qQ/A.

The boundary friction slope (S_f) in Eq. (9) is evaluated from Manning's equation for uniform, steady flow, i.e.,

$$S_f = \frac{n^2|Q|Q}{2.21\ A^2\ R^{4/3}} = |Q|Q/K^2 \quad (10)$$

in which n is the Manning coefficient of frictional resistance, R is the hydraulic radius, and K is the conveyance factor. When the conveyance factor (K) is used to represent S_f, the river channel/valley cross-sectional properties are designated as left floodplain, channel, and right floodplain rather than as a composite channel/valley section. Special orientation for designating left or right is not required as long as consistency is maintained. The conveyance factor is evaluated as follows:

$$K_\ell = \frac{1.49}{n_\ell} A_\ell R_\ell^{2/3} \quad (11)$$

$$K_c = \frac{1.49\ A_c R_c^{2/3}}{n_c\ s_m^{1/2}} \quad (12)$$

$$K_r = \frac{1.49}{n_r} A_r R_r^{2/3} \quad (13)$$

$$K = K_\ell + K_c + K_r \quad (14)$$

in which the subscripts ℓ, c, and r designate left floodplain, channel, and right floodplain, respectively. The sinuosity factors (s_c and s_m) in Eqs. (8), (9), and (12) represent the weighted ratio of the flow-path distance along the floodplains. They vary with depth of flow according to the following relations:

$$s_{c_J} = \frac{\sum_{k=2}^{k=J} \Delta A_{\ell_k} + \Delta A_{c_k} s_{c_k} + \Delta A_{r_k}}{A_{\ell_J} + A_{c_J} + A_{r_J}} \quad (15)$$

$$s_{m_J} = \frac{\sum_{k=2}^{k=J} \Delta K_{\ell_k} + \Delta K_{c_k} s_{m_k} + \Delta K_{r_k}}{K_{\ell_J} + K_{c_J} + K_{r_J}} \quad (15')$$

in which $\Delta A = A_{m+1} - A_m$ and the sinousity factor s_m represents the sinuosity factor for a differential portion of the flow between the m^{th} depth and the $m+1^{th}$ depth. Distances between cross sections are measured along the mean flow path for the floodplain flow. The momentum coefficient for velocity distribution (β) is evaluated as follows:

$$\beta = \frac{1.06\ (K_\ell^2/A_\ell + K_c^2/A_c + K_r^2/A_r)}{(K_\ell + K_c + K_r)^2/(A_\ell + A_c + A_r)} \quad \text{.....(16)}$$

where $\beta = 1.06$ when floodplain characteristics are not specified and the total cross section is treated as a composite section.

The term (S_e) in Eq. (9) is defined as follows:

$$S_e = \frac{k_{ce}\ \Delta(Q/A)^2}{2g\ \Delta x} \quad \text{.....(17)}$$

in which k_{ce} is the expansion-contraction coefficient (Morris and Wiggert, 1972), and $\Delta(Q/A)^2$ is the difference in the term $(Q/A)^2$ at two adjacent cross sections separated by a distance Δx. A provision is made within DAMBRK to automatically change contraction to expansion coefficients and vice versa if the flow direction changes from downstream to upstream in which case the computed Q values are negative. The expansion (k_{ce} = -0.05 to -0.75) or contraction (k_{ce} = 0.05 to 0.4) coefficient is changed to k_n for reverse flows by using the relationship $k_n = -(2.* k_{ce} + 0.1)$ if $k_{ce} > 0$, and $k_n = -(k_{ce} + 0.1)/2$ if $k_{ce} < 0$.

The term (S_i) in Eq. (9) is significant only when the fluid is non-Newtonian. It is evaluated for any non-Newtonian flow as follows:

$$S_i = \frac{\kappa}{\Upsilon}\left[\frac{(b+2)Q}{AD^{b+1}} + \frac{(b+2)\ (\tau_o/\kappa)^b}{2\ D^b}\right]^{1/b} \quad \text{.....(18)}$$

in which Υ is the fluid's unit weight, τ_o is the fluid's yield strength, D is the hydraulic depth (ratio of wetted area to topwidth), b=1/m where m is the power of the power function that fits the fluid's stress-strain properties, and κ is the apparent viscosity or scale factor of the power function. In lieu of actual fluid stress-strain properties, mud/debris flow properties may be estimated from the percent concentration of solids in the fluid (O'Brien and Julien, 1984). An option in DAMBRK allows the term (S_i) to be considered, otherwise S_i is always assumed to be zero.

The active cross-sectional area (A) and inactive (off-channel storage) area (A_o) are obtained from hydrographic surveys and/or topography maps. They are specified as input to DAMBRK as a table of wetted topwidths (B) which vary with elevation at selected cross sections along the channel/valley. Within the model, the topwidth table is integrated using the trapezoidal rule to obtain a table of cross-sectional area versus elevation. Linear interpolation is used for intermediate elevations between specified tabular points. Areas associated with elevations exceeding the maximum value as specified in the table are extrapolated.

The Manning n coefficient is specified for each reach between adjacent cross sections and varies with elevation according to user specified tabular values similar to the topwidths table. Linear interpolation is used for n values associated with intermediate elevations. Values of n for elevations exceeding the tabular elevations are not extrapolated; they are assigned the n value associated with the maximum elevation.

3.2 Solution Technique for Saint-Venant Equations

The expanded Saint-Venant Eqs. (8-9) constitute a system of partial differential equations with two independent variables, x and t, and two dependent variables, h and Q; the remaining terms are either functions of x, t, h, and/or Q, or they are constants. These equations are not amenable to analytical solutions except in cases where the channel geometry and boundary conditions are uncomplicated and the nonlinear properties of the equations are either neglected or made linear. Eqs. (8-9) may be solved numerically by performing two basic steps. First, the partial differential equations are represented by a corresponding set of finite-difference algebraic equations; and second, the system of algebraic equations is solved in conformance with prescribed initial and boundary conditions.

Eqs. (8-9) can be solved by either explicit or implicit finite-difference techniques (Liggett and Cunge, 1975). Explicit methods, although simpler in application, are restricted by mathematical stability considerations to very small computational time steps (on the order of a few seconds for most dam-break waves). Such small time steps cause the explicit methods to be very inefficient in the use of computer time. Implicit finite-difference techniques (Preissmann, 1961; Amein and Fang, 1970; Strelkoff, 1970), however, have no restrictions on the size of the time step due to mathematical stability; however, convergence considerations may require its size to be limited (Fread, 1974a).

Of the various implicit schemes that have been developed, the "weighted four-point" scheme first used by Preissmann (1961), and more recently by Chaudhry and Contractor (1973) and Fread (1974b, 1978) appears most advantageous since it can readily be used with unequal distance steps and its stability-convergence properties can be conveniently controlled. In the weighted, four-point implicit finite-difference scheme, the continuous x-t region in which solutions of h and Q are sought, is represented by a rectangular net of discrete points. The net points are determined by the intersection of lines drawn parallel to the x and t axes. Those parallel to the t-axis represent locations of cross sections; they have a spacing of Δx, which need not be constant. Those parallel to the x-axis represent time lines; they have a spacing of Δt, which also need not be constant. Each point in the rectangular network can be identified by a subscript (i) which designates the x-position and a superscript (j) which designates the particular time line.

The time derivatives are approximated by a forward difference quotient centered between the i^{th} and i+1 points along the x-axis, i.e.,

$$\frac{\partial K}{\partial t} = \frac{K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j}{2\,\Delta t_j} \quad \ldots\ldots(19)$$

where K represents any variable (Q, h, A, A_o, s).

The spatial derivatives are approximated by a forward difference quotient positioned between two adjacent time lines according to weighting factors of θ and $1-\theta$, i.e.,

$$\frac{\partial K}{\partial x} = \theta \left[\frac{K_{i+1}^{j+1} - K_i^{j+1}}{\Delta x_i}\right] + (1-\theta) \left[\frac{K_{i+1}^j - K_i^j}{\Delta x_i}\right] \quad \ldots\ldots(20)$$

Variables other than derivatives are approximated at the time level where the spatial derivatives are evaluated by using the same weighting factors, i.e.,

$$K = \theta \left[\frac{K_i^{j+1} + K_{i+1}^{j+1}}{2}\right] + (1-\theta) \left[\frac{K_i^j + K_{i+1}^j}{2}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

A θ weighting factor of 1.0 yields the fully implicit or backward difference scheme used by Baltzer and Lai (1968). A weighting factor of 0.5 yields the box scheme used by Amein and Fang (1970). The influence of the θ weighting factor on the accuracy of the computations was examined by the author (1974a), who concluded that the accuracy tends to somewhat decrease as θ departs from 0.5 and approaches 1.0. This effect becomes more pronounced as the magnitude of the computational time step increases. Usually, a weighting factor of 0.60 is used so as to minimize the loss of accuracy associated with greater values while avoiding the possibility of a weak or pseudo instability noticed by Baltzer and Lai (1968), and Chaudhry and Contractor (1973) for θ values of 0.5; however, θ may be specified other than 0.60 in the data input to the DAMBRK model via the parameter F1I.

When the finite-difference operators defined by Eqs. (19-21) are used to replace the derivatives and other variables in Eqs. (8-9), the following weighted, four-point implicit, finite-difference equations are obtained:

$$\theta \left[\frac{Q_{i+1}^{j+1} - Q_i^{j+1}}{\Delta x_i}\right] - \theta\, q_i^{j+1} + (1-\theta) \left[\frac{Q_{i+1}^j - Q_i^j}{\Delta x_i}\right] - (1-\theta)\, q_i^j$$

$$+ \left[\frac{s_{c_i}^{j+1}(A+A_o)_i^{j+1} + s_{c_i}^{j+1}(A+A_o)_{i+1}^{j+1} - s_{c_i}^{j}(A+A_o)_i^j - s_{c_i}^{j}(A+A_o)_{i+1}^j}{2\Delta t_j}\right] = 0 \ldots (22)$$

$$\left(\frac{(s_{m_i} Q_i)^{j+1} + (s_{m_i} Q_{i+1})^{j+1} - (s_{m_i} Q_i)^j - (s_{m_i} Q_{i+1})^j}{2\Delta t_j}\right) + \theta \left[\frac{(\beta Q^2/A)_{i+1}^{j+1} - \beta Q^2/A)_i^{j+1}}{\Delta x_i}\right.$$

$$\left. + g\, \bar{A}^{j+1} \left(\frac{h_{i+1}^{j+1} - h_i^{j+1}}{\Delta x_i} + \bar{S}_f^{j+1} + S_e^{j+1} + \bar{S}_i^{j+1}\right)\right] + (1-\theta)$$

$$\left[\frac{(\beta Q^2/A)_{i+1}^j - (\beta Q^2/A)_i^j}{\Delta x_i} + g\, \bar{A}^j \left(\frac{h_{i+1}^j - h_i^j}{\Delta x_i} + \bar{S}_f^j + S_e^j + \bar{S}_i^j\right)\right] = 0 \ldots\ldots\ldots\ldots (23)$$

where:

where:

$$\bar{A} = (A_i + A_{i+1})/2 \quad \text{(24)}$$

$$\bar{S}_f = n^2\ \bar{Q}|\bar{Q}|/(2.2\ \bar{A}^2\ \bar{R}^{4/3}) = \bar{Q}|\bar{Q}|/\bar{K}^2 \quad \text{(25)}$$

$$\bar{Q} = (Q_i + Q_{i+1})/2 \quad \text{(26)}$$

$$\bar{R} = \bar{A}/\bar{B} \quad \text{(27)}$$

$$\bar{B} = (B_i + B_{i+1})/2 \quad \text{(28)}$$

$$\bar{K} = (K_i + K_{i+1})/2 \quad \text{(29)}$$

The term ($\bar{S}_i$) is evaluated using Eq. (18) in which $D = \bar{R}$, $Q = \bar{Q}$, and $A = \bar{A}$. The terms associated with the j^{th} time line are known from either the initial conditions or previous computations. The initial conditions refer to values of h and Q at each node along the x-axis for the first time line (j=1). The initial conditions are further described later in subsection 3.6.

Eqs. (22-23) cannot be solved in an explicit or direct manner for the unknowns since there are four unknowns and only two equations. However, if Eqs. (22-23) are applied to each of the (N-1) rectangular grids between the upstream and downstream boundaries, a total of (2N-2) equations with 2N unknowns can be formulated. (N denotes the total number of nodes or cross sections). Then, prescribed boundary conditions for subcritical flows, one at the upstream boundary and one at the downstream boundary, provide the necessary two additional equations required for the system to be determinate. The boundary conditions are further described later in subsections 3.4 and 3.5. The resulting system of 2N nonlinear equations with 2N unknowns is solved by a functional iterative procedure, the Newton-Raphson method (Amein and Fang, 1970).

Computations for the iterative solution of the nonlinear system are begun by assigning trial values to the 2N unknowns. Substitution of the trial values into the system of nonlinear equations yields a set of 2N residuals. The Newton-Raphson method provides a means for correcting the trial values until the residuals are reduced to a suitable tolerance level. This is usually accomplished in one or two iterations through use of linear extrapolation for the first trial values. If the Newton-Raphson corrections are applied only once, i.e., there is no iteration, the nonlinear system of difference equations degenerates to the equivalent of a quasi-linear, finite-difference formulation of the Saint-Venant equations which may require smaller time steps than the nonlinear formulation for the same degree of numerical accuracy.

A system of 2N x 2N linear equations relates the corrections to the residuals and to a Jacobian coefficient matrix composed of partial derivatives of each equation with respect to each unknown variable in that equation. The Jacobian (coefficient) matrix of the linear system has a banded structure which allows the system to be solved by a compact, quad-diagonal, Gaussian elimination algorithm (Fread, 1971, 1985b), which is very efficient with respect to computing time and storage. The required storage is 2N x 4 and the required number of computational steps is approximately 38N. A more detailed treatment of the solution technique is given elsewhere by the author (1976, 1985b).

When flow is supercritical, the solution technique previously described can be somewhat simplified. Instead of a solution involving 2N x 2N equations, supercritical flow can be solved via a system of only 2 x 2 equations. The unknown h and Q at the upstream section are determined from the two boundary equations. Then, progressing from upstream to downstream in a cascade manner, Eqs. (22-23) are used to obtain h_{i+1} and Q_{i+1} at each section. Since Eqs. (22-23) are nonlinear with respect to h_{i+1} and Q_{i+1}, they are solved by the Newton-Raphson iterative technique applied to a system of two equations with two unknowns. For supercritical flow, this technique has been found to provide a somewhat more stable solution than one involving 2N x 2N equations (Traver 1988).

3.3 Internal Boundaries

There may be locations such as a dam, bridge, or waterfall (short rapids) along a waterway where the Saint-Venant equations are not applicable. At these locations, the flow is rapidly varied rather than gradually varied as necessary for the applicability of the Saint-Venant equations. Empirical water elevation-discharge relations such as weir flow can be utilized for simulating rapidly varying flow. In DAMBRK, unsteady flows are routed along the waterway including points of rapidly varying flow by utilizing internal boundaries. At internal boundaries, cross sections are specified for the upstream and downstream extremities of the section of waterway encompassing the rapidly varying flow. The short reach length between the two cross sections can be any appropriate value from zero to the actual measured distance. Since, as with any other Δx reach, two equations (the Saint-Venant equations) are required, the internal boundary Δx reach requires two equations. The first of the required equations represents the conservation of mass with negligible time-dependent storage, and the second is an empirical, rapidly varied flow equation representing weir, orifice, and/or critical flow. The internal boundary equations are:

$$Q_i = Q_{i+1} \qquad (30)$$

$$Q_i = Q_s + Q_b \qquad (31)$$

in which Q_s and Q_b are the spillway and breach flow, respectively. In this way, the flows Q_i and Q_{i+1} and the elevations h_i and h_{i+1} are in balance with the other flows and elevations occurring simultaneously throughout the entire flow system which may consist of additional dams or bridges which are treated as additional internal boundary conditions via Eqs. (30-31). In fact, DAMBRK can simulate the progression of a dam-break flood through as many as 10 dams and/or bridges in any combination located sequentially along the valley. Any of the dams or bridge-embankments may breach if they are sufficiently overtopped.

3.3.1 Dams

A dam may be considered an internal boundary defined by a short Δx reach between sections i and i+1 in which the flow is governed by Eqs. (30-31). In Eq. (31), the spillway flow (Q_s) is computed from the following expression:

$$Q_s = k_{sp}c_sL_s(h-h_s)^{1.5} + \sqrt{2g}\ c_gA_g(h-h_g)^{0.5} + k_dc_dL_d(h-h_d)^{1.5} + Q_t \quad \ldots\ldots(32)$$

in which k_{sp} is a submergence correction for tailwater effects, c_s is the uncontrolled spillway discharge coefficient, h_s is the uncontrolled spillway crest elevation, c_g is the fixed-gated spillway discharge coefficient, h_g is the center-line elevation of the gated spillway or it is the tailwater elevation if the latter is greater, k_d is a submergence correction for tailwater effects, c_d is the discharge coefficient for flow over the crest of the dam, L_s is the spillway length, A_g is the gate flow area, L_d is the length of the dam crest less L_s and the length of the gates located along the dam crest (L_d may also vary with h according to a specified table of L_d versus h; this allows for dam crests which are not level), and Q_t is a constant (or variable with time) outflow term which is head independent. The uncontrolled spillway flow or the fixed-gated spillway flow can also be represented as a table of head versus discharge values. The gate flow may also be specified as a function of time via a moveable-gate option.

Time-dependent, movable-gate flow (Wortman, 1983) can be simulated with the DAMBRK model by specifying the movable-gate height (H_g) above the gate sill elevation (h_g) and the width of gate opening (W_g) as functions of time. The flow may be either orifice flow and/or weir flow. Weir flow occurs if the gate is not submerged sufficiently or as overtopping flow (Q_{og}) when the reservoir elevation is sufficiently above the top of the dam (h_d). Time-dependent orifice gate flow (Q_g) is computed as follows:

$$Q_g = \sqrt{2g}\ C_o\ W_g\ H_g\ (\hat{h} - H_g/2)^{0.5} + Q_{og} \qquad \text{if } \hat{h} > 1.2\ H_g \quad \ldots\ldots\ldots(33)$$

where:

$$C_o = \frac{0.712}{W_g}\ [W_d-2\ (0.02\ W_d/40 + 0.1)\hat{h}_d]\ (\hat{h}/\hat{h}_d)^{0.1} \qquad 0.60 \le C_o \le 0.72 \quad \ldots\ldots(34)$$

$$\hat{h} = h - h_g \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(35)$$

$$Q_{og} = 3.1\ W_g(h - h_d - H_g)^{1.5} \qquad \text{if } h > h_d + H_g \quad \ldots\ldots\ldots\ldots(36)$$

otherwise, $Q_{og} = 0$. If the tailwater (h_t) is greater than $h_g + H_g$, then $\hat{h}$ in Eq. (35) is the differential head across the gate, i.e., $h - h_t$. Time-dependent weir flow (Q_g) through the gate is computed as follows:

$$Q_g = Q_d \left[1 - (1 - H_g/\hat{h})^{1.5}\right] (\hat{h}/\hat{h}_d)^{1.6} \qquad \text{if } H_g \le \hat{h} \le 1.2\, H_g \quad \ldots\ldots(37)$$

where:

$$Q_d = 3.9\, k_g \left[W_d - 2\,(0.02\, W_d/40 + 0.1)\, \hat{h}_d\right] \hat{h}_d^{\,1.5} \quad \ldots\ldots(38)$$

$$\hat{h} = h - h_g \quad \ldots\ldots(39)$$

$$\hat{h}_d = h - h_g \qquad \text{at } t = 0 \quad \ldots\ldots(40)$$

$$W_d = W_g \qquad \text{maximum for all } t \quad \ldots\ldots(41)$$

$$Q_g = Q_d\, (\hat{h}/\hat{h}_d)^{1.6} \qquad 0 < \hat{h} < H_g \quad \ldots\ldots(42)$$

$$k_g = 1.0 - 27.8\left[(h_t - h_g)/\hat{h} - 0.67\right]^3 \qquad \text{if } (h_t - h_g)/\hat{h} > 0.67 \quad \ldots\ldots(43)$$

otherwise $k_g = 0$. There will be some error in the computed flow when the gate is narrow, i.e., small W_g relative to H_g. Transition from orifice flow to weir flow may produce a slight discontinuity. The gate loss coefficient will usually be in the range of 0.65 to 0.70.

The breach outflow (Q_b) is computed as broad-crested weir flow, i.e.,

$$Q_b = c_v k_s \left[3.1\, b_i\, (h-h_b)^{1.5} + 2.45\, Z\, (h-h_b)^{2.5}\right] \quad \ldots\ldots(44)$$

in which c_v is a small correction for velocity of approach, b_i is the instantaneous breach bottom width as described by Eq. (3), h is the elevation of the water surface just upstream of the structure, h_b is the elevation of the breach bottom which is assumed to be a function of the breach formation time as described by Eq. (2), Z is the side slope of the breach, and k_s is the submergence correction due to the downstream tailwater elevation (h_t), i.e.,

$$k_s = 1.0 - 27.8\left[\frac{h_t-h_b}{h-h_b} - 0.67\right]^3, \text{ if } (h_t-h_b)/(h-h_b) > 0.67 \quad \text{.............(45)}$$

otherwise, $k_s = 1.0$. Eq. (45) is also used to evaluate k_{sp} and k_d where k_s, h_b are replaced by k_{sp}, h_g and k_d, h_d, respectively. Eq. (45) was developed by the author from a graphical representation by Venard (1954). The velocity of approach correction factor is computed from the following (Brater, 1959):

$$c_v = 1.0 + 0.023\, Q_i^2/[B_d^{\,2}\,(h-h_{bm})^2(h-h_b)] \quad \text{..............................(46)}$$

in which B_d is the reservoir width at the dam and h_{bm} is the terminal elevation of the breach botton. If the breach is formed by piping, Z is assumed zero (rectangular shape) and Eq. (44) is replaced by an orifice equation, i.e.,

$$Q_b = 4.8\, A_p(h-\bar{h})^{1/2} \quad \text{..(47)}$$

where:

$$A_p = 2b_i\,(h_p-h_b) \quad \text{...(48)}$$

in which h_p is the specified center-line elevation of the pipe, and $\bar{h}=h_p$ or $\bar{h} = h_t$ if $h_t > h_p$. The breach flow ceases to be orifice flow and becomes broad-crested weir flow when the reservoir elevation (h) lowers sufficiently and/or the pipe enlarges sufficiently that:

$$h < 3\,h_p - 2\,h_b \quad \text{..(49)}$$

3.3.2 Bridges

Highway/railway bridges and their associated earthen embankments which are located anywhere within the routing reach may be treated also as internal boundary conditions. Eqs. (30-31) are used at each bridge; the term Q_s in Eq. (31) is computed by the following expression:

$$Q_s = \sqrt{2g}\ C\ A_{br}(h_i - h_{i+1} + V^2/2g - \Delta h_f)^{1/2} + cc_u\ L_u\ k_u$$

$$(h_i - h_{cu})^{3/2} + cc_\ell\ L_\ell\ k_\ell(h_i - h_{c\ell})^{3/2} \quad \text{.............................(50)}$$

in which,

$$k_u = 1.0, \qquad \text{if } h_{ru} \leq 0.76 \quad \text{.............(51)}$$

$$k_u = 1.0 - c_u(h_{ru} - 0.76)^3, \qquad \text{if } h_{ru} > 0.76 \quad \text{.............(52)}$$

$$c_u = 133(h_{ru} - 0.78) + 10, \qquad \text{if } 0.76 < h_{ru} \leq 0.96 \quad \text{......(53)}$$

$$c_u = 400(h_{ru} - 0.96) + 34, \qquad \text{if } h_{ru} > 0.96 \quad \text{.............(54)}$$

$$h_{ru} = (h_{i+1} - h_{cu})/(h_i - h_{cu}) \quad \text{...(55)}$$

$$cc_u = 3.02(h_i - h_{cu})^{0.015}, \qquad \text{if } 0 < h_u \leq 0.15 \quad \text{..........(56)}$$

$$cc_u = 3.06 + 0.27(h_u - 0.15), \qquad \text{if } h_u > 0.15 \quad \text{..............(57)}$$

$$h_u = (h_i - h_{cu})/w_u \quad \text{...(58)}$$

$$\Delta h_f = \Delta x_i\ (Q_{br}/\bar{K}_i)^2 \quad \text{..(59)}$$

$$Q_{br} = \sqrt{2g}\ C\ A_{br}\ (h_i - h_{i+1} + V_i^{\ 2}/2g)^{1/2} \quad \text{.............................(60)}$$

$$V = Q_i/A_i \quad \text{...(61)}$$

in which C is a coefficient of bridge flow which accounts for piers, alignment, etc. (see Chow, 1959), A_{br} is the cross-sectional flow area of the bridge opening at section i+1 (downstream end of bridge) which is specified via a tabular relation of wetted topwidth versus elevation, h_{cu} is the elevation of the upper embankment crest, h_i is the water surface elevation at section i (slightly upstream of bridge), h_{i+1} is the water surface elevation at section i+1, V is the velocity of flow within the bridge opening, L_u is the length of the upper embankment crest perpendicular to the flow direction including the length of bridge at elevation h_{cu} (L_u may be specified as a tabular relation with elevation), k_u is the submergence correction factor for flow over the upper embankment crest, and w_u is the width (parallel to flow direction) of the crest of the upper embankment. In Eq. (50), the terms with an (ℓ) subscript refer to a lower embankment crest and these terms are defined by Eqs. (51-58) in which the (u) subscripts are replaced with (ℓ) subscripts. Eqs. (51-58) were developed by the author from basic information on flow over road embankments as reported by the U.S. Dept. of Transportation (1978). A breach of the embankment is treated the same as a dam breach in that Q_b in Eq. (31) is computed via Eqs. (44-49) which now pertain to the bridge embankment. When the bridge opening becomes submerged, C in Eqs. (50) and (60) is reduced to C' for orifice flow according to the following:

$$C' = c_o C \qquad (62)$$

where:

$$c_o = 1.0 - (r - 0.09) \qquad \text{if } 0.09 \le r \le 0.31 \qquad (63)$$

otherwise, $c_o = 1.0$, and ....(64)

$$r = (h_i - h_{br})/d_i \qquad (65)$$

in which h_{br} is the elevation of the bottom of the bridge deck, and d_i is the flow depth at section i located a distance w_u upstream of the upstream face of the bridge. The DAMBRK model creates a table of A_{br} from specified tabular values of the width of bridge opening versus elevation; the highest elevation specified for the table is h_{br} and at this elevation the width must be zero.

The cross section designated by i+1 should represent the downstream end of the bridge opening; if the bridge opening is small enough to cause critical flow, the section can be moved slightly downstream where the section properties represent the channel rather than the constricted bridge opening. This will cause some small error in the computation of Q_s in Eq. (50) since the term h_{i+1} should represent the water surface elevation at the constricted section. A contraction coefficient (k_{ce}) should be specified for the Δx reach upstream of section i and an expansion coefficient should be specified for the Δx reach downstream of section i+1.

3.3.3 Waterfalls or Rapids

If a short reach of the river contains a waterfall or steep rapids which will not be completely submerged at high flows due to downstream backwater effects, the DAMBRK model can simulate the critical flow through the falls or rapids by considering them to be an internal boundary represented by a dam. A rating is used for the spillway flow where h_{sp} specifies the invert elevation of the upstream or control section of the channel at the beginning of the falls or rapids. The specified rating table of discharge versus water surface elevation may be computed from the following equation for critical flow:

$$Q = (gA^3/B)^{0.5} \quad \text{...} (66)$$

Of course, the breach parameters associated with a dam are not applicable in this case and should be specified with zero values.

3.4 Upstream Boundary

The upstream boundary is required to obtain a solution of the Saint-Venant equations. In most applications of the DAMBRK model, this is simply a specified discharge hydrograph, i.e.,

$$Q_1 = QI(t) \quad \text{...} (67)$$

in which Q_1 is the flow at section 1 (the most upstream cross section), and QI(t) represents the specified flow at time (t). The hydrograph values, QI(t), are specified at either constant or variable time intervals. Discharges are linearly interpolated from the table of discharge versus time. If the upstream flow is steady, i.e., it is constant for all time, the specified discharge table has the same discharge specified for all times. Generally, the upstream flow should not be zero. Also, the upstream hydrograph should be specified for the total duration of time that the Saint-Venant equations are to be solved.

If the water surface of the most upstream reservoir is assumed to remain level as it varies with time due to the inflows and spillway/breach outflows, then the following boundary equation is used:

$$Q_1 = QI(t) - 0.5\ \bar{S}_a\ 43560.\ \Delta h/\Delta t \quad \text{.....................................(68)}$$

in which Q_1 is the discharge at the upstream most section (the upstream face of the dam), QI(t) is the specified inflow to the reservoir, $\bar{S}_a$ is the average surface area (acre-ft) of the reservoir during the Δt time interval, and Δh is the change in reservoir elevation during the time step. Eq. (68) represents a level-pool routing algorithm in the form of an upstream boundary condition. The use of Eq. (68) requires that a table of reservoir surface area versus elevation be specified.

If the flow is supercritical at the upstream end of the routing reach, i.e.,

$$Fr_1 = V_1/\sqrt{gA_1B_1} \geq 1 \quad \text{...(69)}$$

two boundary equations are used at the upstream section. The first is Eq. (67) and the second is the following stage-discharge relation:

$$Q_i = \frac{1.49}{n_i} A_i R_i^{2/3} S^{1/2} = K_i S^{1/2} \quad \text{..(70)}$$

in which,

$$S = (h_i - h_{i+1})/\Delta x_i \quad \text{..(71)}$$

and i=1, the most upstream cross section.

3.5 Downstream Boundary

When the flow near the downstream extremity of the routing reach is subcritical, i.e.,

$$Fr_N = V_N / \sqrt{g\ A_N/B_N} < 1 \quad \text{..(72)}$$

where N designates the number of the most downstream cross section, a known relationship between flow and depth or depth and time must be specified. Depending on the physical characteristics of the downstream section, the DAMBRK model allows the appropriate specification of one of the following four downstream boundary equations:

(1) Single-value rating:

$$Q_i = Q(h) \quad \text{...} (73)$$

in which Q(h) represents a specified tabular relation of Q and h, and i = N.

(2) Generated dynamic loop-rating:

$$Q_i = \frac{1.49}{n_i} A_i R_i^{2/3} S^{1/2} = K_i S^{1/2} \quad \text{...................................} (74)$$

where:

$$S = (h_{i-1} - h_i)/\Delta x_{i-1} + (Q_i' - Q_i)/[0.5g (A_i + A_{i-1}) \Delta t]$$

$$+ (Q_{i-1}^2/A_{i-1} - Q_i^2/A_i)/[0.5g (A_i + A_{i-1}) \Delta x_{i-1}] \quad \text{....................} (75)$$

in which Q_i' is the discharge at time (t - Δt).

(3) Critical flow rating:

$$Q_i = \sqrt{g} A_i^{3/2}/B_i^{1/2} \quad \text{...} (76)$$

(4) Water level time series:

$$h_i = h(t) \quad \text{...} (77)$$

in which h(t) represents a specified time series of water elevation versus time (t).

If channel control exists, i.e., the flow at section N is controlled by the channel properties, then either Eq. (73) or Eq. (74) can be selected.

Eq. (73) is useful if an empirical Q(h) relation is available which is essentially single-valued, i.e., for each water surface elevation there is only one discharge. When a known Q(h) relation does not exist, then the dynamic loop-rating, Eq. (74) may be used. The loop-rating allows two water surface elevations to exist for each discharge value. On the rising limb of the hydrograph, the water surface elevation is less that that which occurs for the same discharge on the recession limb. The magnitude of the loop is directly proportional to the rate of increase in water surface elevation and inversely proportional to the invert slope of the channel. Thus for the rapidly rising hydrographs associated with dam-break floods, the loop-rating is more likely to be significant; although if the channel slope is quite steep (say 100 ft/mi or greater), the loop will probably be less than 0.5 feet. The following can be used to estimate the magnitude of the loop (Fread, 1973):

$$\Delta h = 2D \left[1 - (S_o/S)^{0.3}\right] \qquad (78)$$

where:

$$S = S_o + \delta h \left[\frac{0.52\ n}{S_o^{1/2}\ D^{2/3}} + \frac{0.01\ S_o^{1/2}\ D^{2/3}}{D\ n}\right] \qquad (79)$$

in which D is the hydraulic depth (ft), S_o is the channel bottom slope (ft/ft), δh is the rate of rise of the water elevation (ft/sec), and Δh is the magnitude of the loop (ft). The dynamic loop-rating, Eq. (74), may be subject to numerical instability when the channel bottom slope is less than about one ft/mi. In this situation, the downstream boundary can be relocated a sufficient distance further downstream of the original boundary location. Errors in Q and h due to the alternate use of a single-value rating (which is not subject to numerical problems) are damped-out in the vicinity of the original boundary location where computed Q and h values are of interest. A channel control boundary, Eq. (73) or Eq. (74), should not be located where changes in flow further downstream can affect the flow at the chosen boundary location, e.g., just upstream of where a significant tributary flow enters, or upstream within the backwater effect of a bridge, dam, or tidal fluctuation.

A critical-flow rating may be used where there is a natural waterfall or short steep rapids which are not completely drowned-out at high flows due to either natural or man-made flow controls downstream of the rapids.

A specified water-level time series may be used when the downstream boundary is located in a wide estuary or bay where the water surface elevation is controlled only by the tidal fluctuation and not by the flow emanating from the upstream routing reach. Also, this boundary condition can be used when the channel terminates in a large lake whose level is not appreciably influenced by the incoming flow. In this case, the water surface elevation is specified as a constant value for all time during the simulation.

A single-value rating, Eq. (73), may also be used when the downstream boundary is a dam where the total flow through the dam is controlled by the water surface elevation occurring immediately upstream of the dam and not by the water elevation downstream of the dam due to tailwater submergence conditions.

There are some cases where a dam constitutes the downstream boundary; in these, the DAMBRK model uses the internal boundary condition, Eq. (31), as the downstream boundary condition. These cases are described later in subsection 3.24 as options 4, 5, 6, 9, and 10.

3.6 Initial Conditions

In order to solve the unsteady flow equations, the state of the flow (h and Q) must be known at all cross sections at the beginning (t=0) of the simulation. This is known as the initial condition of the flow. The DAMBRK model assumes the flow to be steady, nonuniform flow where the flow at each cross section is initially computed as:

$$Q_i = Q_{i-1} + q_{i-1}\,\Delta x_{i-1} \qquad i=2,3,\ldots N \quad \ldots\ldots\ldots\ldots\ldots\ldots(80)$$

where Q_1 is the known steady discharge at t=0 at the dam, i.e., the upstream boundary of the downstream valley, and q_i is any specified lateral inflow at t=0 from tributaries existing between the specified cross sections spaced at intervals of Δx along the valley. The steady discharge at t=0 is usually assumed to be nonzero, i.e., an initially dry downstream channel is <u>not</u> usually simulated in DAMBRK. An exception to this must be used when mud/debris flows are routed. This is described later in subsection 3.13 pertaining to mud/debris flow routing. A nonzero initial flow is not an important restriction, especially when maximum flows and peak stages are of paramount interest in the dam-break flood analysis. The tributary lateral inflow must be specified by the user throughout the simulation period. If these flows are relatively small compared to the dam-break flood, they may be omitted in the simulation.

The water surface elevations associated with the steady flow also must be computed at t=0. If the flow is subcritical, this is accomplished by using the iterative Newton-Raphson method to solve the following backwater equation for h_i:

$$(Q^2/A)_{i+1} - (Q^2/A)_i + g\,\bar{A}_i\,(h_{i+1} - h_i + \Delta x_i \bar{S}_f + \Delta x_i \bar{S}_i) = 0 \quad \text{............(81)}$$

in which $\bar{A}$, $\bar{S}_f$, and $\bar{S}_i$ are defined by Eqs. (24), (25), and (18) respectively. Eq. (81) is a simplified form of the momentum Eq. (9) where the first term is taken as zero for steady flow, and L' is assumed to be zero. The computations proceed in the upstream direction (i = N, N-1, ..., 3, 2, 1). The starting water surface elevation (h_N) can be obtained from the specified downstream boundary condition for either a discharge of Q_N or the elevation h_N at t=0. When the generated dynamic loop-rating, Eq. (74), is used as the downstream boundary, there can be some numerical difficulties due to errors associated with h_N. The Manning equation, i.e.,

$$Q_N = 1.49/n_N A_N R_N^{2/3} S^{1/2} = K_N S^{1/2} \quad \text{....................................(82)}$$

is used to compute h_N. Eq. (82) is solved iteratively for h_N using the Newton-Raphson method. The energy slope (S) is approximated by using the channel bottom slope (S_o) associated with the most downstream Δx reach; however, this may not be a sufficiently accurate approximation resulting in an erroneous value for h_N which then produces subsequent errors in the computed values for h_i via Eq. (81). The erroneous initial conditions result in fluctuations in the discharges and elevations as the Saint-Venant finite-difference Eqs. (22-23) are applied. Thus, it may appear that some unsteady flows are occurring in the vicinity of the downstream boundary long before the floodwave actually has reached that location; these arise as the Saint-Venant solution attempts to correct the erroneous initial conditions. This type of numerical noise may be minimized or possibly eliminated by a judicious change in the invert elevations of the two most downstream cross sections such that another value of S_o is used to better approximate S. The true initial energy slope (S) can be estimated from the first run of DAMBRK wherein it is the stabilized water surface slope in the vicinity of the downstream boundary obtained after several time steps and before the arrival of the floodwave. Another numerical problem occurs when the value for S_o used to approximate S in Eq. (82) is negative due to the invert at section N-1 being less than that at section N. This will cause the program to stop since a negative value cannot be raised to a power on the computer. This problem can be overcome by adjusting the invert elevations of the two most downstream sections such that S_o is positive, i.e.,

$$S_o = (h_{N-1} - h_N)/\Delta x_{N-1} > 0 \quad \text{.......................................} (83)$$

If the flow is supercritical, the computations for h_i proceed from upstream to downstream (i = 1, 2, 3, ..., N-1, N). In this case, Eq. (81) is used to compute h_{i+1}. The starting water surface elevation (h_1) is obtained by using Eq. (82) with N replaced by 1 and Eq. (83) with N replaced by 2. Additional details concerning the solution of Eqs. (81) and (82) can be found elsewhere (Fread, 1985b).

3.7 Mixed (Subcritical/Supercritical) Flow

In previous versions of DAMBRK, numerical difficulties usually resulting in aborted computer runs occurred when the flow changed from subcritical to supercritical or vice versa. This was caused by the fact that the Saint-Venant equations are not applicable to such transition flows passing through critical depth. If the flows were either subcritical or supercritical for all cross sections throughout the duration of the simulation, the previous version of DAMBRK could properly compute the unsteady flow; however, the user was required to designate if the flow were subcritical or supercritical by assigning the input variable (KSUPC) a value of 0 for subcritical or 1 for supercritical. Of course, the Froude number (Fr) can be used to determine if the flow is subcritical or supercritical; however, a more convenient a priori predictor is:

$$S_c = 77000\ n^2/D^{1/3} \quad \text{.......................................} (84)$$

in which S_c is the critical slope (ft/mi), n is the Manning coefficient and D is the hydraulic depth (A/B). Comparison of S_c with the channel bottom slope (S_o) is a good indicator of the type of flow, i.e.,

$S_o > S_c$ supercritical flow ....(85)

$S_o < S_c$ subcritical flow ....(86)

An inspection of Eq. (84) indicates the magnitude of S_c is directly and strongly dependent on n while inversely and weakly dependent on D. Hence, usually overbank flow with increased flow resistance due to trees, etc. require steeper slopes for supercritical flow to occur than flow within the channel bank (bankfull) with smaller flow resistance even though D is greater for the higher flows. Also, from Eq. (84) it is evident that a moderate increase in the n value may cause the flow to change from supercritical to subcritical. In many applications, the flow is supercritical for low flows within bankfull and changes to subcritical flow as the flow increases and inundates the floodplain. Another common situation encountered when applying the DAMBRK model is when the roughness coefficient is specified as essentially constant for all flow depths and the bottom slope is such that the low flows are subcritical while high flows become supercritical as D increases. Therefore, in many applications, elimination of the mixture of subcritical/ supercritical flow could be accomplished by making minor changes in the estimated n values, yet within the bounds of uncertainty associated with the n values. In other cases, if the changes in the n values proves to be excessive, the total routing reach could be divided into a number of reaches where the flow is entirely subcritical or supercritical within each reach. Then the DAMBRK model could be used, via separate applications, to route the flow reach by reach from upstream to downstream. In other situations, if supercritical flow occurred only in a few isolated short and steep reaches, these could be modeled via a critical flow rating of discharge versus elevation and each short reach could be considered an internal boundary or dam with a rating curve based on critical flow through the upstream section of the steep reach.

From the preceding discussion, it is evident that many of the numerical difficulties encountered because of the occurrence of mixed (subcritical/

supercritical) flow could be overcome through judicious changes to the n values or creative approaches in using the DAMBRK model.

3.7.1 New Mixed-Flow Algorithm

The '88 version of the DAMBRK model contains an alternative method for treating the problem of mixed flow. It consists of an algorithmic procedure which automatically subdivides the total routing reach into sub-reaches in which only subcritical or supercritical flow occurs. The transition locations where the flow changes from subcritical to supercritical or vice versa are treated as boundary conditions thus avoiding the application of the Saint-Venant equations to the transition flow. This method has previously been described by the author (1983, 1985b); however, this is the first time the mixed-flow algorithm has been available in the DAMBRK model. The user may choose the mixed-flow algorithm by specifying either 2, 3, or 4 for the KSUPC parameter. The mixed-flow algorithm increases computer run times by about 20 percent.

The mixed-flow algorithm consists of two components, one for obtaining the initial condition of discharge and water surface elevation at t=0 and another which functions during the unsteady flow solution.

The initial condition component, which is similar to that described by Molinas and Yang (1985), uses the same method of determining the initial flow at each cross section as described previously in subsection 3.6. The water elevations are obtained by the following algorithm: (1) normal and critical depths are obtained for each section -- the section is designated subcritical if normal depth is greater than critical depth or it is designated supercritical if normal is less than critical after a check is made to see if downstream elevations created by a dam may drown-out the supercritical depths existing upstream; (2) commencing at the downstream boundary, a backwater solution proceeds from a known elevation (dependent on the downstream boundary condition at t=0) in an upstream direction until supercritical flow occurs or if supercritical flow occurs at the downstream boundary, the computations proceed in the downstream direction from the normal depth at the upstream-most section of all contiguous sections having supercritical flow; (3) when internal boundaries, such as a dam, are encountered, the specified water elevations occurring at t=0 for each reservoir are used for the backwater solution or if a bridge is encountered, Eq. (50), is solved iteratively until the correct value of h_i is determined from known values of Q_i and h_{i+1}; bridges are allowed to exist within a supercritical reach; however, dams must have at least two upstream sections having subcritical flow; (4) steps (2) and (3) are repeated as necessary until the water surface elevations for all sections have been obtained.

The time-dependent component uses the Froude number of the estimated flow occurring at each cross section to group contiguous sections into subcritical sub-reaches and supercritical sub-reaches. Contiguous sections with a Froude number less than or equal to 0.95 are grouped into subcritical sub-reaches and those with a Froude number greater that or equal to 1.05 are grouped into supercritical sub-reaches. Those sections with Froude numbers between 0.95 and 1.05 are considered critical sections. However, isolated critical sections that are surrounded by subcritical sections are grouped with a subcritical sub-reach, while isolated critical sections amongst supercritical sections are grouped with a supercritical sub-reach. The upstream and downstream limits of the subcritical/supercritical reaches are noted and used to determine the range over which the Saint-Venant finite-difference equations are applied. During a Δt time step, the solution commences with the most upstream sub-reach and proceeds sub-reach by sub-reach in the downstream direction. The upstream and downstream boundary conditions for each sub-reach are selected according to the following algorithm: (1) if the most upstream reach is subcritical, the upstream boundary is Eq. (67) and the downstream boundary is Eq. (76) since flow must pass through critical when the next downstream sub-reach is supercritical; (2) if the most upstream reach is supercritical, the upstream boundary is Eq. (67) and a downstream boundary is not required for the supercritical reach since flow disturbances created downstream of the supercritical reach cannot propagate upstream into the supercritical reach; (3) if an inner sub-reach (a sub-reach which is neither the most upstream nor the most downstream sub-reach) is supercritical, the following equations are used for the two upstream boundary equations:

$$Q_1 = Q'(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (87)$$

$$y = y'(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (88)$$

in which $Q'(t)$ is the most recently computed flow at the last cross section of the upstream subcritical sub-reach and $y'(t)$ is the computed critical water surface elevation of the downstream most (critical section) of the upstream subcritical sub-reach; (4) if an inner sub-reach is subcritical, Eq. (87) is used for the upstream boundary in which $Q'(t)$ represents the computed flow at the last section of the upstream supercritical sub-reach and the critical flow Eq. (76) is used as the downstream boundary; (5) if the most downstream sub-reach is subcritical, Eq. (87) is used for the upstream boundary condition and the downstream boundary condition is appropriately selected from Eqs. (73-77) by the user; (6) if the most downstream sub-reach is supercritical, Eqs. (87-88) are used as the upstream boundary equations and no downstream boundary is required.

A hydraulic jump occurs between the last section of a supercritical sub-reach and the first section of the adjacent downstream subcritical sub-reach, although an equation for such is not directly used. To account for the possible upstream movement of the jump the following procedure is utilized before advancing to the next time step: (1) the subcritical elevation (h_e) is extrapolated to the adjacent upstream supercritical section; (2) the sequent water surface elevation of the adjacent upstream supercritical section is iteratively computed via the bi-section method applied to the following sequent elevation equation:

$$\frac{Q^2}{gA} + \bar{z}A - \frac{Q^2}{gA'} - \bar{z}'A' = 0 \quad \text{.......................................(89)}$$

in which $\bar{z}$ is the distance from the water surface to the center of gravity of the wetted cross section, A is the wetted area, Q is the computed flow at the section, and the superscript (') represents variables associated with the sequent elevation (h') while the variables with no superscript are associated with the supercritical elevation; (3) if the sequent elevation (h') is greater than the extrapolated elevation (h_e), the jump is not moved upstream; however, if $h' \leq h_e$, the jump is moved upstream section by section until $h' > h_e$.

To account for the possibility of the jump moving downstream (if it did not move upstream), the following procedure is utilized before advancing to the next time step: (1) starting at the most upstream section of the subcritical sub-reach, the supercritical elevation is computed using Eq. (81) and its sequent elevation (h') is computed by applying the iterative bi-section method to Eq. (89); (2) using the most recently computed subcritical elevation (h), if $h \geq h'$, the jump is not moved downstream; however, if $h < h'$, the jump is moved downstream section by section until $h \geq h'$.

Sub-reaches wherein the flow is essentially critical can cause some numerical difficulties when the new mixed-flow algorithm is used to locate possible movement of the jump. In those cases, it is recommended to <u>not</u> allow the jump to move by choosing KSUPC = 4. When KSUPC = 2 or KSUPC = 3, the mixed-flow algorithm allows for possible movement of the hydraulic jump. Use of KSUPC = 3 rather than KSUPC=2 is recommended for greater numerical robustness of the mixed-flow algorithm wherein the Θ weighting factor in the Saint-Venant finite-difference Eqs. (22-23) is set to 1.0 for the supercritical reaches only; otherwise, it is always defaulted to a value of 0.6 unless specified otherwise by the user via the input parameter (F1I). Also, when KSUPC = 3, the jump is allowed to move only if the Froude number is greater than 2.0.

Smaller computational distance steps (Δx) are required in the vicinity of the transition reaches between subcritical and supercritical flow. This is particularly required both upstream and downstream of a critical flow section to avoid numerical difficulties. Smaller Δx reaches also will enable more accurate location of hydraulic jumps. A very convenient feature for controlling the computational distance step size is available within DAMBRK and is described later in subsection 3.18.

During the computation of initial conditions for mixed flows, the movement of a hydraulic jump from the position determined by comparison of normal and critical elevations is not considered. However, since the DAMBRK model always uses the Saint-Venant equations to improve the initial conditions before the unsteady solution commences, the jump at t = 0 is allowed to move upstream or downstream from its original location via the technique described previously in the time-dependent component of the mixed-flow algorithm.

A reservoir, which has a sufficiently steep slope for supercritical flow to occur in its upper reaches as the reservoir pool is lowered by the breach outflow, may be treated as entirely subcritical flow by assigning sufficiently large Manning n values for the lower elevations of each reservoir cross section. The required n values can be determined via Eqs. (84-86). Specification of such n values in the lower portions of the reservoir generally will not significantly affect the computed outflow hydrographs.

3.8 Lateral Flows

Unsteady flows associated with tributaries upstream or downstream of a dam or anywhere along the routing reach can be added to the unsteady flow resulting from the dam failure. This is accomplished via the term q in Eq. (8) or (22). The tributary flow is distributed along a single Δx reach. Lateral inflows are specified as a time series of total discharge Q(t) occurring along the Δx reach. Within the DAMBRK model, Q(t) is divided by Δx to obtain q(t). Backwater effects of the dam-break flow on the tributary flow are ignored, and the tributary flow is assumed to enter perpendicular to the dam-break flow. Outflows are assigned negative values. Outflows which occur as broad-crested weir flow over a levee or natural crest may be simulated within the DAMBRK model, i.e.,

$$q = -C_w \, (\bar{h} - h_w)^{1.5} \quad \ldots\ldots(90)$$

in which C_w is the discharge coefficient for broad-crested weir flow $(2.6 \leq C_w \leq 3.2)$, $\bar{h}$ is the average of the computed water elevations at sections i and i+1 bounding the Δx reach in which the weir outflow occurs, and h_w is the average crest elevation of the weir along the Δx reach. The crest elevation, discharge coefficient, and location along the river/valley must be specified by the user.

3.9 Routing Losses

Often in the case of dam-break floods, where the extremely high flows inundate considerable portions of overbank or floodplain, a measurable loss of flow volume occurs. This is due to infiltration into the relatively dry overbank material and flood detention storage losses due to topographic depressions and/or water trapped behind field irrigation levees. Such losses of flow may be taken into account via the term q in Eq. (8) or (22). An expression describing the loss is given by the following:

$$q_m = -0.00458\ V_L\ P/(L\ \overline{T}) \qquad (91)$$

in which V_L is the outflow volume (acre-ft) from the reservoir; P is the volume loss ratio expressed as a decimal percent; L is the length (mi) of downstream channel where it is assumed that the loss occurs uniformly along the length; and $\overline{T}$ is the distance weighted average duration (hr) of the flood wave throughout the reach length L; and q_m is the maximum lateral outflow (cfs/ft) occurring along the routing reach L throughout the duration of flow. The mean lateral outflow is proportioned in time and distance along the reach L such that $q_i^j=0$ when $Q_i^j=Q_i^1$ and $q_i^j=q_m$ when $Q_i^j=Q_{max_i}$. Thus:

$$q_i^j = \frac{(Q_i^j-Q_i^1)}{(Q_{max_i}-Q_i^1)}\ q_m \qquad (92)$$

$$Q_{max_i} = Q_{max_N} + (Q_{max}-Q_{max_N})\ \left(\frac{X_N - X_i}{L}\right)^{m_e} \qquad (93)$$

where Q_i^1 is the initial flow, Q_{max_i} is the estimated maximum flow at each i^{th} cross section, Q_{max_N} is the maximum routed discharge at the downstream section (X_n), Q_{max} is the maximum discharge at the dam, and m_e is a fitted exponent. The parameter P may vary from only a few percent to as much as 30, depending on the condition of the downstream valley. The parameters (q_m, Q_{max_N}, and m_e) must be specified by the user. The losses should occur throughout the entire routing reach for which the model is applied. The use of this feature promotes less conservative flooding results and should be used only when past experience with the particular river/valley provides a good estimate for the parameter (P) in Eq. (91). The exponent (m_e) in Eq. (93) is obtained by iteratively fitting Eq. (93) to previous DAMBRK simulations using trial m_e values. Also, the final value for Q_{max_N} in Eq. (93) is obtained through the same iterative procedure.

3.10 Floodplain Compartments

The DAMBRK model can simulate the exchange of flow between the river and one or more floodplain compartments. The floodplain compartments are formed by levees or road embankments which run parallel to the river on either or both sides of the river, and other levees or road embankments which run perpendicular to the river. All compartments must be contiguous to the river. Flow transfer between a floodplain compartment and the river is assumed to occur along one or more Δx reaches which adjoin the river and a floodplain compartment; this flow is assumed to be broad-crested weir flow with submergence correction. Flow can be either away from the river or into the river, depending on the relative water surface elevations of the river and the floodplain compartment. The river elevations are computed via Eqs. (22-23), and the floodplain water surface elevations are computed by a simple storage routing relation, i.e.,

$$V_\ell^{\,t} = V_\ell^{\,t-\Delta t} + \left(I^t - O^t\right) \Delta t/43560 \qquad (94)$$

in which V_ℓ is the volume (acre-ft) in the floodplain compartment at time t or t-Δt depending on the water elevation, I is the inflow from the river and/or adjacent floodplain compartments, and O is the outflow from the floodplain compartment to the river and/or to adjacent floodplain compartments. Flow transfer between adjacent floodplain compartments is also controlled by broad-crested weir flow with submergence correction. The broad-crested weir flow into or out of a single compartment is determined according to the following:

$$I = c_f \, s_b \left(h_r - h_w\right)^{3/2} \qquad \text{if } h_r > h_w \text{ and } h_r > h_{fp} \qquad (95)$$

$$O = c_f \, s_b \left(h_{fp} - h_w\right)^{3/2} \qquad \text{if } h_{fp} > h_w \text{ and } h_{fp} > h_p \qquad (96)$$

in which c_f is a specified discharge coefficient, h_r is the river elevation, h_{fp} is the water surface elevation of the floodplain, and s_b is the submergence correction factor, i.e.,

$$s_b = 1.0 - 27.8 \left(H_r - 0.67\right)^3 \qquad \text{if } H_r > 0.67 \qquad (97)$$

$$H_r = \left(h_r - h_w\right)/\left(h_{fp} - h_w\right) \qquad \text{if } h_r > h_w \text{ and } h_r > h_{fp} \qquad (98)$$

$$H_r = (h_{fp} - h_w)/(h_r - h_w) \qquad \text{if } h_{fp} > h_w \text{ and } h_{fp} > h_r \qquad (99)$$

otherwise, S_b = 1.0, and h_w is the specified elevation of the crest of the levee. The floodplain elevation (h_{fp}) is obtained iteratively via a table look-up algorithm applied to the specified table of volume-elevation values. The outflow from a floodplain compartment may also include that from one or more pumps associated with each floodplain compartment. Each pump has a specified discharge-head relation given in tabular form along with start-up and shut-off operation instructions which depend on specified water surface elevations. The pumps discharge to the river.

3.11 Landslide-Generated Waves

Reservoirs are sometimes subject to landslides which rush into the reservoir, displacing a portion of the reservoir contents and, thereby, creating a very steep water wave which travels up and down the length of the reservoir (Davidson and McCartney, 1975). This wave may have sufficient amplitude to overtop the dam and precipitate a failure of the dam, or the wave by itself may be large enough to cause catastrophic flooding downstream of the dam without resulting in the failure of the dam as in the case of a concrete dam in Vaiont, Italy in 1963.

The capability to generate waves produced by landslides is provided within DAMBRK. The volume of the landslide mass, its porosity, and the time interval over which the landslide occurs are specified as input to the model. In the model, the landslide mass is deposited during very small computational time steps within the reservoir in layers commencing at the center of the reservoir and extending toward the side of the landslide, and simultaneously the original dimensions of the reservoir are reduced. The time rate of reduction in the reservoir cross-sectional area (Koutitas, 1977) creates the wave during the solution of the unsteady flow Eqs. (8-9), which are applied to the reservoir cross sections which describe the reservoir storage characteristics. The upstream boundary condition is given by Eq. (67),and the downstream boundary condition is given by Eq. (73) or (31). The initial conditions are obtained as described by Eqs. (80-81) for steady nonuniform flow.

Wave runup is not considered in the model. For near vertical faces of concrete dams the runup may be neglected; however, for earthen dams the usual angle of the earthfill on the reservoir side will result in a surge that advances up the face of the dam to a height approximately equal to 2.5 times the height of the landslide-generated wave (Morris and Wiggert, 1972).

3.12 Pressurized Flow

The DAMBRK model may be used to simulate unsteady flows which can change from free-surface flow to pressurized flow from one section to another and/or as the flow changes with time. When the flow passing through a section of closed conduit of any shape completely submerges the section, the flow properties change from those of free-surface to pressurized flow. In the latter type of flow, disturbances in the flow are propagated at velocities many times greater than those for free-surface flow. A technique which enables the Saint-Venant equations to properly simulate pressurized flow is included within DAMBRK. It follows the method first described by Cunge and Wegner (1964) and recently described by the author (1984c) for application of the Saint-Venant equations to unsteady flows in a network of storm sewers. In this method, closed conduits are specified by a table of topwidth versus elevation in a manner similar for open channels such as rivers, except when the topwidth diminishes to zero at the top of the closed conduit it is actually specified to have a very small topwidth (b^*) which extends vertically upward for at least one or more feet. Within DAMBRK this topwidth is extrapolated for elevations larger than the last specified elevation; hence, the extrapolated topwidth is always b^* for all elevations since the last two specified topwidths are each b^*. Thus, by expressing the topwidth table in this manner for closed conduits, the Saint-Venant equations properly simulate either free-surface or pressurized flow. The celerity ($\hat{c}$) of disturbances sensed by the Saint-Venant equations is given by the following:

$$\hat{c} = \sqrt{g\ A/B} \quad \text{...} (100)$$

in which $B = b^*$ for flows in which the free-surface water elevations extend above the top of the closed conduit, as is the case for pressurized flow. Of course, B represents the actual wetted topwidth for those sections experiencing free surface flow. An inspection of Eq. (100) shows that $\hat{c}$ may become quite large as B becomes very small. The value of b^* may be obtained from Eq. (100) if $\hat{c}$ for pressurized flow is known from previous observations or b^* can be computed from conduit and water properties as delineated by the author

(1984c). Thus, in DAMBRK, flows may be simulated which are always free surface in some reaches where the sections are open while in other reaches with closed conduit sections, the flow may be initially pressurized or with time change from free surface to pressurized flow and vice versa.

3.13 Mud/Debris Flows

The DAMBRK model may be used to route specified hydrographs of mud/debris flows or the non-Newtonian contents of a mine-tailings dam by using Eq. (18) to define S_i (viscous dissipation term) in Eq. (9). A control parameter (MUD) is used to activate this capability which requires the user to specify the parameters κ (viscosity), τ_o (initial shear strength), m (exponent in power function equation). Eq. (18) was derived from principles of viscous flow in which the shear stress τ_s was expressed as a power function of the non-Newtonian fluid's stress-rate of strain (dv/dy) relation, i.e.,

$$\tau_s = \tau_o + \kappa \, (dv/dy)^m \quad \text{.......................................(101)}$$

Eq. (101) represents an empirical fit of a power function equation to measured fluid properties. If this is not available, κ, τ_o, and γ (unit weight) may be roughly approximated for debris flows from empirical relationships using only the fluid's percent solids concentration (O'Brien and Julien, 1984).

Eq. (101) may be transformed via double integration with respect to y (depth) to an expression for the velocity, i.e.,

$$V = \left(\frac{\gamma S}{\kappa}\right)^b \frac{D^{b+1}}{b+2} - \left(\frac{\tau_o}{\kappa}\right)^b \frac{D}{2} \quad \text{.......................................(102)}$$

in which b = 1/m. If Eq. (102) is analyzed on the basis of steady flow with S approximated by the channel bottom slope (S_o), it can be seen that for the velocity to be positive the first term on the right-hand-side must exceed the second. Thus, if b=1, i.e., a Bingham plastic fluid, then the following inequality must be satisfied in order for flow to occur:

$$D > \frac{1.5\ \tau_o}{\gamma\ S_o} \quad \text{.......................................(103)}$$

Hence, for moderate values of S_o (say $S_o < 0.01$) and large τ_o, the hydraulic depth (D) must be quite large. For example, if S = 0.005 ft/ft, γ = 120 lb/ft^3, κ = 20 lb-sec/ft (the true dynamic viscosity) and τ_o = 10 lb/ft^2, D must be greater than 25 ft for flow to occur. Such large depths required for the initiation of flow necessitates the ability to route mud/debris flows through a channel that is initially dry. A dry channel condition is not usually required for Newtonian fluids such as water waves due to dam failures since the depth associated with an arbitrarily small initial flow is insignificant compared to the flood peak.

Since mud/debris flows may require dry-bed routing capability, this option is provided in DAMBRK via the control parameter (IWF) which allows the use of an approximation technique for tracking the tip of the flood wave as it advances downstream. The wave-tip velocity is approximated as the computed velocity just upstream of the tip. The solution net or x-dimension of the x-t solution plane is expanded in the downstream direction at the velocity of the wave tip, i.e., cross sections are added in the downstream direction as the wave propagates downstream. Of course, the cross sections required to describe the channel/valley as far downstream as desired are specified by the user, as well as, the computational distance step parameter (DXM). The net is expanded as the wave tip reaches a specified computational point. Therefore, the ratio of computational distance step to the time step should approximately equal the wave-tip velocity. The tip velocity can be initially estimated via Eq. (102) with D obtained from Eq. (103). The wave-tip velocity can be improved through a few iterative applications of the DAMBRK model.

If a mine-tailings dam failure is simulated, the non-Newtonian contents of the dam do not behave significantly different than water as far as the use of the broad-crested weir coefficients for the dam-breach outflow is concerned. Thus, within DAMBRK, the weir coefficients for breach flow are not changed for mine-tailings dam breaches.

The mud/debris flow capability within the DAMBRK model is controlled by the parameter (MUD), which allows the selection of the following solution methodologies: (1) if MUD = 0, no mud/debris flow capabilities are used, i.e., S_i in Eq. (9) is set to zero; (2) if MUD = 1, the specified mud/debris-flow hydrograph is routed through a very steep channel reach via a nonlinear iterative Muskingum-Cunge algorithm (a brief description follows); (3) if

MUD = 2, the Muskingum-Cunge method is used for a very steep reach of channel and the routed hydrograph, i.e., the computed hydrograph at the downstream end of the steep channel reach, is then routed through a moderate to flat reach of channel using the Saint-Venant Eqs. (8-9); (4) if MUD = 3, the specified mud/debris-flow hydrograph is routed through the channel routing reach using the Saint-Venant Eqs. (8-9); and (5) if MUD = 4, the specified hydrograph is routed through two reaches as in MUD = 2 except the Saint-Venant Eqs. (8-9) are used for both reaches.

3.13.1 Muskingum-Cunge algorithm for very steep reaches

The DAMBRK model allows a specified mud/debris flow hydrograph to be routed through a very steep channel reach using the Muskingum-Cunge method (Cunge, 1969; Ponce and Yevjevich, 1978; Ponce, 1981). This method has been modified to allow the routing coefficients to be nonlinear which requires an iterative solution procedure to be utilized. The modified Muskingum-Cunge algorithm for computing the discharge at each i^{th} cross section as $i=2,3,\ldots,N$ is:

$$Q_i^{j+1} = C_1\, Q_{i-1}^{j} + C_2\, Q_{i-1}^{j+1} + C_3\, Q_i^{j} + C_4 \quad \text{(104)}$$

where:

$$C_1 = \left[(1 - \theta)\, \overline{C}\, \Delta t/\Delta x + X\right]/C_5 \quad \text{(105)}$$

$$C_2 = (\theta\, \overline{C}\, \Delta t/\Delta x - X)/C_5 \quad \text{(106)}$$

$$C_3 = \left[1 - (1 - \theta)\, \overline{C}\, \Delta t/\Delta x - x\right]/C_5 \quad \text{(107)}$$

$$C_4 = \left[\theta\, (q_i^j + q_i^{j+1})\, \overline{C}\, \Delta t\right]/C_5 \quad \text{(108)}$$

$$C_5 = 1 + \theta\, \overline{C}\, \Delta t/\Delta x - X \quad \text{(109)}$$

$$\overline{C} = K_w\, \overline{Q}/\overline{A} \quad \text{(110)}$$

$$\overline{Q} = (Q_{i-1}^{j} + Q_{i-1}^{j+1} + Q_i^{j} + Q_i^{j+1})/4 \quad \text{(111)}$$

$$\bar{A} = (A_{i-1}^{j} + A_{i-1}^{j+1} + A_{i}^{j} + A_{i}^{j+1})/4 \quad \text{.....(112)}$$

$$K_w = 5/3 - 2/3\ (\bar{D}\ d\bar{B}/dh)/\bar{B} \quad \text{.....(113)}$$

$$\bar{B} = (B_{i-1}^{j} + B_{i-1}^{j+1} + B_{i}^{j} + B_{i}^{j+1})/4 \quad \text{.....(114)}$$

$$\bar{D} = \bar{A}/\bar{B} \quad \text{.....(115)}$$

$$X = 0.5\ [1 - \bar{D}/(K_w S_o \Delta x)] \quad \text{.....(116)}$$

The discharge at the upstream boundary (i=1) is obtained from Eq. (67). The depth associated with the flow (Q) at the downstream boundary is computed from the uniform mud/debris flow equation which is simply the product of A times the velocity as given by Eq. (102), i.e.,

$$Q = \left(\frac{\gamma S}{\kappa}\right)^{b} \frac{AD^{b+1}}{b+2} - \left(\frac{\tau_o}{\kappa}\right)^{b} \frac{AD}{2} \quad \text{.....(117)}$$

where:

$$S = S_o - S_f \quad \text{.....(118)}$$

$$S_f = n^2 Q^2/(2.2\ A^2 R^{4/3}) \quad \text{.....(119)}$$

$$R = A/P \quad \text{.....(120)}$$

$$D = A/B \quad \text{.....(121)}$$

in which A, B, D, $d\bar{B}/dh$, P, and n are known functions of the fluid elevation (h) of the cross-section at the downstream boundary. The elevation (h) for any interior point is obtained by solving the backwater equation (81). Since Q_i^{j+1} is unknown in Eq. (111), it is estimated at the first iteration by extrapolation of flows occurring at previous times; thereafter the estimated value computed at the previous iteration is used. Convergence is attained when the following criterion is satisfied:

$$|Q_i^{j+1} - \hat{Q}_i^{j+1}| < 0.01\ Q_i^{j+1} \quad \text{.......................................} (122)$$

in which $\hat{Q}_i^{j+1}$ is the estimated value.

The Muskingum-Cunge method should not be used for routing dam-break waves. Nevertheless, its range of applicability for routing slower rising flood waves can be extended through first solving the Muskingum-Cunge equation (Eq. (104)) for discharge and then stages via the backwater equation (Eq. (81)) iteratively. With an expected routing error of 5 percent or less, the improved Muskingum-Cunge backwater routing method can be used for hydrographs with time of rise (T_r, in hrs) greater than $3.65/S_m^{0.4}$, where S_m is the channel slope (ft/mi). The Δx and Δt values should be selected according to similar criteria for the Saint-Venant Eqs. (8-9) as described later in subsections (13.19) and (13.20).

3.14 Conveyance Option

The friction slope (S_f) may be evaluated by two different methods as indicated by Eq. (10). The first method (composite option) directly utilizes composite values of the Manning n, A, and R while the second method indirectly uses separate Manning n, A, and R values for the channel and left and right portions of the floodplain. In the second method, (conveyance option), the conveyances for each portion of the total cross section are computed initially within DAMBRK via Eqs. (11-13) and then the total conveyance for a particular section is obtained by summing the separate conveyances as in Eq. (14). The conveyance option is activated when the user assigns the control parameter (KFLP) a value of unity. Leaving KFLP blank or assigning it a zero value results in the composite representation of S_f.

An advantage in using the conveyance option is the elimination of numerical convergence difficulties associated with the composite option for S_f; this occurs when the cross-sectional geometry consists of a channel and a very wide and flat floodplain. In this case, the derivative (dB/dh), which is necessary to evaluate when using the Newton-Raphson iterative technique to solve the Saint-Venant equations, is not well-defined in the vicinity of the top of bank where the topwidth greatly increases with a small increase in elevation. The total conveyance function, which also varies with elevation, is well behaved, i.e., the slope dK/dh is smoothly varying in the vicinity of the top of bank whereas the slope dB/dh is somewhat discontinuous in this same region. The selection of the conveyance option requires the separate specification of the topwidths and n values for each of the portions of the total cross section, i.e., channel, left floodplain, and right floodplain. The left and/or the right floodplain properties (B and n) may be specified as zero values if there is no floodplain or portion thereof.

3.15 Sinuosity Factor

A meandering or sinuous channel provides a longer flow path than that provided by the floodplain. This effect is simulated in DAMBRK via the sinuosity factor (s) in Eqs. (8-9). The cross sections are designated via a mileage parameter (XS) which is measured along the mean flow path through the floodplain. The sinuosity factor which is always ≥ 1.0 is the ratio of the flow-path distance along the meandering channel to the mean flow-path distance (XS) along the floodplain. The sinuosity factor is specified for each reach between two adjacent specified cross sections, and it may decrese for elevations extending above the top of bank. For those elevations used to describe the topwidth at bankfull elevation and below, the sinuosity factor is as previously defined; however, at elevations above bankfull, the sinuosity factor for each layer of flow between specified elevations is decreased. This trend is continued until for those flow layers, say 5 to 10 feet above bankfull, the sinuosity factor is reduced to unity which indicates that the floodplain flow has fully captured the upper layers of flow directly above the channel. The sinuosity factor may be left blank when specified for DAMBRK, in which case it is automatically set to unity for all elevations. This represents a straight river with no meanders. When the conveyance option is <u>not</u> used, i.e., KFLP = 0, the sinuosity factors are omitted from the data input; however, they are then automatically set to unity within DAMBRK.

The sinuosity factor as used in the finite-difference, Saint-Venant Eqs. (22-23) is depth-weighted within DAMBRK according to Eq. (15). The depth-weighting results in a sinuosity factor which only approaches unity, even for the upper elevations associated with large floodplain flows. This occurs since the total flow is still comprised of the relatively small flow within bankfull which follows the meandering channel, as well as, the larger portion of the total flow which follows the floodplain flow path.

3.16 Hydraulic Radius Option

The hydraulic radius (R) used in Eqs. (10-13) is normally evaluated within DAMBRK as A/B or the hydraulic depth (D). This is satisfactory for almost all river channels since A/B = A/P, where P is the wetted perimeter. For narrow, deep channels this approximation is not as good. Therefore, an option to use R = A/P is provided in DAMBRK by assigning a value of unity to the control parameter, KPRES. When this option is selected, the wetted perimeter (P) is computed from the specified topwidth (B_k) versus elevation (H_k) table according to the following trapezoidal approximation:

$$P_1 = B_1 \quad \text{...} (123)$$

$$P_k = P_{k-1} + 2 \left[0.25 \left(B_k - B_{k-1}\right)^2 + \left(H_k - H_{k-1}\right)^2\right]^{0.5} \qquad k = 2,3,.. \quad .....(124)$$

in which the k subscript designates the particular level within the cross section. The invert (bottom) of the section is designated with k = 1.

3.17 Reservoir Cross-Section Option

Cross-sectional information in the form of topwidth (B_k) versus elevation (H_k) for reservoirs is often difficult if not impossible to obtain; however, most reservoirs have been described with a surface area (S_{a_k}) - elevation table. An option within DAMBRK automatically creates the necessary topwidth-elevation tables from a specified surface area-elevation table. This enables the application of the Saint-Venant equations to simulate the reservoir hydraulics with the same specified data as the more approximate level pool treatment of the reservoir. The increased accuracy of the Saint-Venant equations is most appropriate when the reservoir is long and the reservoir inflow hydrograph is large with a rapidly rising limb. Also, as the time of failure (τ) of a breached dam decreases to very small values (say less than 1 minute), the Saint-Venant equations are able to simulate the negative wave formation within the reservoir.

The cross sections created within DAMBRK for the reservoir are based on the following assumptions: (1) the reservoir has a pyramidal shape, i.e., a pyramid turned on its side where its base forms the dam and its height represents the length of the reservoir; (2) the bottom slope of the reservoir is uniform and equal to h_d/RLM, where RLM is specified by the user as the length of the reservoir in miles; (3) the reservoir is proportioned into three equal length regions with four cross sections along the length of the reservoir and into four depth strata; the surface area of the reservoir is assumed to be proportional along the entire length of the reservoir at the upper strata according to 0.28, 0.21, 0.14 and 0.03 factors for each section commencing with the one nearest the dam and proceeding upstream; the next lower strata has three regions with 0.33. 0.25, and 0.17 proportionality factors; the next lower strata consists of two regions with 0.57 and 0.43 proportionality factors; and the lowest strata consists of only one region equal to RLM/3 in length and having a proportionality factor of 1.0; (4) the width (B) of a cross section is computed according to the following trapezoidal relation:

$$B_{k,\ell} = 2\ p_{k,\ell}\ S_{a_k}\ 43560./(RLM * 5280./3) \quad \text{.............................(125)}$$

in which $p_{k,\ell}$ is the proportionality factor, S_a is the reservoir surface area (acres), RLM is the reservoir length (miles), the subscript k denotes strata, and the subscript ℓ denotes each one of the four cross sections along the reservoir length including the one immediately upstream of the dam; and (5) Manning n values of 0.025 are automatically specified for the two regions nearer the dam, and 0.030 is specified for the upstream region. Computational reach lengths Δx_i (miles) are automatically determined as follows:

$$\Delta x_i = RLM/(3\ N_{\ell\ell}) \qquad \ell\ell = 1,\ 2,\ 3 \quad \text{..............(126)}$$

in which $N_1 = 0.55\ N_R$, $N_2 = 0.3\ N_R$, $N_3 = N_R - (N_1 + N_2)$ and N_R is specified by the user as the total number of computational reaches to be used within the reservoir. The option to create reservoir cross sections from the surface area-elevation table is controlled by specifying a negative value for the reach number (IDAM) identifying the location of the dam, i.e., IDAM(1) is specified as ($-N_R$). Also, the reservoir length (RLM) and the surface area-elevation table must be specified. This option is applicable for only the most upstream reservoir.

3.18 Cross-Section Interpolation

Within DAMBRK is an option to generate additional cross sections between any two adjacent specified cross sections. The properties of the additional cross sections are linearly interpolated. Both active and inactive (off-channel storage) cross-sectional properties are generated via the interpolation procedure. Generation of the additional cross sections enables the computational distance step (Δx_i) used to solve the finite-difference Saint-Venant Eqs. (22-23) to be smaller than the distance step separating the original specified cross sections. The original distance steps are determined by considerations for properly specifying the river/valley volume via the cross sections with an assumption of linear variations between specified sections. Thus, the river/valley sections are located at narrow and wide sections with linear variation from one to the other. Original specified sections are also selected according to special features that need to be described, e.g., bridges, dams, locations where significant changes in the channel bottom slope or the Manning n occur, locations where lateral inflow/outflow occur, and locations where information about flood wave properties is desired.

The option for interpolation of cross sections requires adherence to the following criteria when specifying the cross sections: (1) all cross sections should have the same number of topwidths; (2) if possible, the bankfull topwidth should be in the same relative position of the topwidth table for all sections, e.g., the second topwidth could represent the bankfull width for all sections.

The computational distance step (Δx_i) is controlled within DAMBRK by the parameter (DXM_i) which is specified for each (i^{th}) original distance step between specified cross sections. The relation of DXM_i to Δx_i is simply $\Delta x_i = 5280\ DXM_i$ in which DXM is the computational reach length (mi) and Δx_i is the computational distance step (ft). Criteria for specifying DXM_i are given in the following subsection 3.19.

If interpolated sections are created between two adjacent specified cross sections and lateral inflows or computed outflows as described in subsection

3.8 or those associated with floodplain compartments as described in subsection 3.10, the following provisions automatically occur: (1) for lateral inflows, the inflow is made to occur entirely within the most upstream (Δx_i) computational distance step within the original reach between specified sections; (2) for computed lateral outflows, the computed outflow occurs for each of the computational distance stops (Δx_i) each having the same crest elevation (h_w) as specified for the original reach between specified sections; and (3) for computed outflow to floodplain compartments, the computed flow occurs for each of the (Δx_i) computational distance steps each having the same levee crest elevation (h_w) as specified for the original reach.

3.19 Selection of Computational Distance Steps

It is most important that computational distance steps (Δx_i) in the finite difference Saint-Venant Eqs. (22-23) be properly selected via the parameter (DXM_i) in order to avoid computational difficulties. When the computational distance step is chosen too large, the resulting truncation error [the difference between the true solution of the partial differential Saint-Venant Eqs. (8-9) and the approximate solution of the finite-difference Saint-Venant Eqs. (22-23)] may be so large that the computed solution of h_i and Q_i is totally unrealistic, e.g., the computed flow depths have negative values. This causes the DAMBRK program to abort when a negative depth or cross-sectional area is raised to a power which is necessary when computing the friction slope (S_f) via Eq. (25). Large truncation errors can also cause irregularities in the computed hydrograph as manifested by spurious spikes in the rising and/or falling limbs. Three criteria are used to select the computational distance steps.

The first of the criteria is related to contracting/expanding cross sections. Samuels (1985) found that the four-point implicit difference scheme theoretically requires the following criteria be satisfied within any computational distance step:

$$0.635 < A_{i+1}/A_i < 1.576 \quad \ldots\ldots (127)$$

Basco (1987) found from numerical experimental studies using DAMBRK that the factor 0.635 should be increased to 0.70 and the factor 1.576 could be increased to 2.00. Within DAMBRK, the following algorithm automatically selects the computational distance step (Δx_i) such that the above contraction/expansion criterion, Eq. (127), is conservatively satisfied:

$$DXM_i = L/M \quad\text{.......................................}(128)$$

where:

$$M = 1 + 2 \,|A_i - A_{i+1}|/\hat{A} \quad\text{.......................................}(129)$$

$$\hat{A} = A_{i+1} \qquad \text{if } A_i > A_{i+1} \quad \text{contraction ....}(130)$$

$$\hat{A} = A_i \qquad \text{if } A_i < A_{i+1} \quad \text{expansion ....}(131)$$

in which L is the original distance step (mi) between specified cross sections and M is rounded to the nearest smaller integer which represents the new number of computational distance steps within the original distance step.

The <u>second</u> of the three criteria is the following:

$$DXM_i \le c\, t_r/(2M) \quad\text{.......................................}(132)$$

in which DXM_i is the computational distance step (mi), c is the wave speed (mi/hr), t_r is the time of rise (hr) of the routed hydrograph, and $5 \le M \le 20$ (default to M = 10). The selection of the time step is discussed in the next subsection. The wave speed for most unsteady flows, including dam-break floods, that propagate through a river/valley can be approximated by the kinematic wave velocity, i.e.,

$$c = 0.68\, K_w\, V \quad\text{.......................................}(133)$$

where K_w is given by Eq. (113) and V is computed from the Manning equation, i.e.,

$$V = 1.49/n\; R^{2/3}\; S_o^{1/2} \quad\text{.......................................}(134)$$

Within DAMBRK, there are three options for selecting the computational distance step (DXM). They are: (1) the DXM value is left blank and the value used in the routing computations is determined within DAMBRK; (2) DXM is determined for each reach using Eqs. (132-134) and a negative sign (-) preceding the first value signals the model to bypass the computation of DXM; and (3) DXM is entered (specified) with a positive value which signals the model to compute DXM and print it out, while still using the specified value for the routing. When the DAMBRK model computes the DXM value, the wave speed is determined via the technique described by the author (1987b) and Wetmore and Fread (1983) which is used in the NWS Simplified Dam-Break model (SMPDBK). This technique computes the maximum breach outflow using Eq. (7), and then routes the peak through the downstream river/valley using dimensionless routing relationships which were previously developed via the DAMBRK model for a myriad of scenarios consisting of various sizes of dams, reservoirs, breaches, downstream valleys, and different valley slopes and roughness factors. This technique neglects backwater effects from downstream natural constrictions and dams or bridges. Hence, when downstream backwater effects may be significant, the computed wave speed used in option (2) or (3) may result in computed DXM values somewhat smaller than those obtained from Eq. (132) if a more accurate wave speed were used. Once a DAMBRK solution has been obtained, better values of DXM conforming to Eq. (132) may be used in subsequent solutions.

The third criterion for selecting the computational distance step is related to significant changes in the channel bottom slope (Sm, ft/mi). Wherever the channel bottom slope (Sm_i) abruptly changes, smaller computational distance steps say 1/5 to 1/2 of those required by the second criteria, Eq. (132), are automatically determined. Also, wherever the flow changes from subcritical to supercritical or vice versa, the computational distance step (DXM_i) should be smaller. The DAMBRK model automatically determines computational distance steps according to the following approximations: If $Sm_i > 30$ and $Sm_i/Sm_{i+1} > 2$, then

$$DXM_i = Sm_{i+1}/Sm_i \quad \text{..........} (135)$$

If $Sm_i < 30$, $Sm_{i+1} > 30$, and $Sm_{i+1}/Sm_i > 2$, then

$$DXM_i = Sm_i/Sm_{i+1} \quad \text{..........} (136)$$

Abrupt changes in bottom slope where computational distance steps are too large not only cause numerical difficulties when solving the Saint-Venant Eqs. (22-23) but also cause numerical difficulties when solving Eq. (81) to obtain the initial water surface elevations.

The option to automatically select the DXM_i values should not be relied on for all situations. Judicious selection of DXM_i values by the user provides a means for intelligent use of the DAMBRK model for unusual or complex applications.

3.20 Selection of Computational Time Steps

Equally important to the computational distance steps (Δx_i) are the computational time steps (Δt_j). Their proper selection prevents the occurrence of numerical difficulties due to excessive truncation errors in the finite-difference approximate solution of the Saint-Venant equations. Also, if the computational time steps are too large, the specified hydrograph or the hydrograph generated by the breaching of the dam will not be accurately characterized, i.e., if the time steps are too large, the peak of the hydrograph can be ignored as the time steps (Δt_j) step through and actually bypass the hydrograph peak. To ensure small truncation errors and to properly treat the hydrograph peak, the following criteria are used within DAMBRK: (1) the selected time step Δt_j is evenly divisible into the smaller of either the time of rise of the specified hydrograph or the time of failure (τ) of the breach; usually the latter is sufficiently small such as to also cause the Δt_j time step to coincide with the peak of the specified hydrograph; (2) the time of rise (t_r) of the specified hydrograph or the time of failure of the breach is divided by a factor (M), where $5 \leq M \leq 20$; usually a value of 20 is sufficiently large to produce computational time steps sufficiently small so as to minimize truncation errors.

Within the DAMBRK model, there is an option to automatically select the computational time step when the input parameter DTHM is specified as blank or 0.0. In this option, the model uses the following computational time step:

$$\Delta t_j = t_r/M \quad \text{...(137)}$$

in which the subscript j designates the particular time line from 1 to the total number of time lines used during the simulation, t_r is the time of rise of the specified hydrograph and M=20 until the breach is just about to begin to form. Thereafter, the time step is given by

$$\Delta t_j = \tau/M \quad \text{...} (138)$$

in which τ is the breach time (hr) of formation and M=20. If there is only one dam being simulated, then the computational time step is allowed to increase as the time of rise of the breach hydrograph (τ) increases due to dispersion of the wave as it propagates downstream. In applications with two or more dams, the time step is not allowed to increase as the wave propagates downstream; however, in Eq. (138) the minimum τ for any of the dams which have commenced their breach formation is used to compute Δt_j.

When DTHM is specified as a positive value equal to the computational time step size, i.e., DTHM > 0, the DAMBRK model uses this computational time step throughout the period of simulation. When DTHM is specified with a negative sign (-) preceding its value, Eqs. (137-138) are used to determine the computational time step with M=|DTHM|; this allows the user to have some control over the size of the variable time step.

Another parameter (TFI) can also be specified to allow control of the time step size. In this case, DTHM is specified as the computational time step size which is used by the DAMBRK model until the simulation time exceeds the specified value of TFI, at which time the model uses Eq. (138) to compute the new computational time step.

When selecting the computational time step, Eqs. (137-138) can be used along with a suitable value of M. Also, the author has derived a theoretical relation for the computational time step which may be used in selecting DTHM. This relation is:

$$\Delta t_j \leq 0.075\ c\ t_r\ (\hat{Z}/D)^{0.5} \quad \text{...} (139)$$

where:

$$\hat{Z} = (1 - \varepsilon^2)/[4\theta^2\varepsilon^2 - (2\theta - 2)^2] \quad (140)$$

in which Δt_j is the computational time step (hrs), c is the wave speed (mi/hr), t_r is the time of rise of the flood wave (hrs), D is the average hydraulic depth (ft), ε is the permissible error ratio ($0.90 < \varepsilon < 0.99$) of the approximate finite-difference solution to the true solution, and θ is the finite-difference weighting factor.

Generally, with a smaller computational time step, there is a smaller truncation error (if Eq. (132) is used to select the computational distance step) and there is less chance for the occurrence of numerical difficulties; however, the smaller the time step, the smaller the distance step must be, and this results in considerably more computer time needed to obtain the solution. In fact, halving the time step requires halving the distance step which then quadruples the required computer time. Thus, there is always a trade-off between accuracy or an absence of numerical difficulties and the required expenditure of computational time.

It is recommended that the computational time step (Δt_j) either be specified by the model user or computed automatically within the DAMBRK model. Then, the computational distance step (Δx_i) may either be specified by the user or computed automatically within the model.

3.21 Robust Computational Features

There are two features within the DAMBRK model that help maintain computational robustness and prevent numerical difficulties in addition to the previously mentioned subcritical/supercritical algorithm, conveyance representation of the friction slope (S_f), and distance and time step selection criteria. Many simulations which would normally abort are computed successfully because of the following two computational features.

The first feature is a "safety net" or numerical low-flow filter which prevents computed values of h_i and Q_i from retaining values which are <u>not</u> possible according to a limiting assumption of the type of hydrographs that may be specified and/or created within DAMBRK and which are routed via the Saint-Venant equations. Under this assumption, all hydrographs are not allowed to have flow values less than the initial flow at t=0. Thus, any computed flow or elevation during the simulation that is smaller than the initial flow or elevation at each i^{th} section is considered to be erroneous due to the trunction error in the approximate Saint-Venant difference solution. Such computed flows or elevations are set to their previous value before the last computations were made. This prevents the retention of critical errors in depth and flow in the vicinity of a rapidly rising wave front such as associated with dam-break waves or any sudden discharge releases. These errors are usually manifested as flows and elevations less than the initial flow through which the steep wave propagates. In fact the erroneous elevations may even be lower than the channel bottom elevation and cause the computer simulation to abort as a negative area or hydraulic radius is raised to a power such as in the friction slope computation given by Eq. (10). These errors usually can be controlled by proper selection of the computational distance and time steps; however, the low-flow filter permits somewhat larger computational steps which provide savings in computer simulation time and storage. The numerical filter or safety net may be decommissioned by specifying the parameter F1I = 0.50. This will allow the following hydraulic phenomena to occur: (1) the specified hydrograph or generated breach hydrograph can have flows that are less than the initial flow at t=0, and (2) the computed flows may reverse direction and propagate upstream, in which case the flows have negative (-) values.

The second feature within DAMBRK that assists in providing computational robustness is the automatic reduction in the computational time step when the numerical solution of the finite-difference, Saint-Venant equations fails to converge within 9 iterations. If nonconvergence occurs, i.e., if at any computational section either ε_{h_i} or ε_{Q_i} are greater than their respective allowable tolerances, then the computation is repeated with a reduced time step of 1/2 of the original time step; subsequently at 1/8, then at 1/16, if nonconvergence persists. If nonconvergence still occurs, then the θ weighting factor is increased by 0.2 and the computations repeated with computational time step of one-half the original time step. This is repeated until θ is equal to 1.0 at which time the computations proceed to the next full time step assuming the most recently computed values are correct although convergence was not attained. If at any time during the previously described iteration procedure convergence is attained, the computations proceed to the next time level using a time step equal to the difference between the original and that which caused convergence and a Θ weighting factor of 0.6 unless specified otherwise by the user via the parameter F1I. (As previously described in subsection 3.19, the F1I also can be used to remove the low-flow filter or safety net by specifying its value as 0.5 which signifies to the DAMBRK model to use the value of 0.6 for Θ.) At any time during the computations, if nonconvergence occurs and the time step is reduced, this can be printed out to notify the user of this situation (JNK $\geq$ 5, where JNK is an output control parameter). This does not constitute an invalid solution; on the contrary, a successful solution is attained whenever the time step is advanced forward in time with the solutions of h_i and Q_i obtained in less than the allowable 9 iterations. Often, computational difficulties can be overcome via one or two time step reductions. However, if the solution advances forward in time with nonconvergence occurring and the Θ value has been increased to a value of one, then the solution is suspect and all h_i and Q_i should be closely examined at that particular time line to see if the results appear reasonable. Usually, if final nonconvergence occurs, i.e., the Θ factor has been increased to a value of one, the simulation should be repeated with appropriate data modifications, e.g., the computational distance steps should be adjusted to more closely satisfy Eq. (132).

3.22 Off-Channel (Dead) Storage

At each cross section, a portion of the section may not convey flow; this is designated to be inactive flow area or off-channel (dead) storage. The portion of the cross section which conveys flow is termed the active cross section. In the Saint-Venant Eqs. (8-9), the active area is designated as A and the inactive area as A_o. Every cross section must have active area; however, a section need not have inactive area. The presence of inactive area is subjectively determined by discerning those portions of a cross section where large eddies may occur and the flow is not directed in the downstream direction. Therein, the flow is temporarily stored as the water elevation rises to inundate those portions of the cross section, yet little if any quantity of flow is conveyed to other sections located further downstream.

Off-channel storage is often associated with contracting and expanding sections. Streamlines tend to be more flexible as flow contracts, hence less off-channel storage is associated with a contracting reach than with an expanding reach where large eddies are easily formed by the streamlines as they gradually expand from a contracted section to a wider downstream section.

Another instance of off-channel storage occurs when flow temporarily stores within the downstream reaches of a tributary which connects to the river through which the flood is being routed via the Saint-Venant equations. In this case, the off-channel storage width (BSS) is zero or nonexistent at a section on the river coincident with the upstream bank of the tributary and also the BSS is zero at a section of the routed river coincident with the downstream bank of the tributary. However, a section located along the river and midway between the other two sections does have an off-channel storage width (BSS). This value may be determined from the following relation:

$$BSS_k = 2\ (43560)\ Sa_k/L \qquad (141)$$

in which the subscript k designates the particular elevation within the cross section (the elevation is usually associated with topographic contour elevations), Sa_k represents the surface area (acres) of that portion of the tributary which would be inundated at the k^{th} elevation due to the backwater pool caused by the flow in the river, L is the length (ft) of the reach along the routed river bounded by the two banks (upstream and downstream) of the connecting tributary, and BSS is the width (ft) of the off-channel portion of the cross section along the routed river and coincident with the middle of the tributary. Of course, if flow is occurring in the tributary due to runoff from its upstream drainage basin, then that portion of the tributary section needed to convey this flow should <u>not</u> be included in the determination of BSS_k, i.e., only those elevations exceeding that required to convey the tributary flow should be used to compute Sa_k in Eq. (141). When off-channel storage areas are used in the Saint-Venant equations, it is implied that as the water surface elevation rises at the center of the river cross section, that same elevation is attained within the computational time step interval throughout limits of the specified off-channel storage area associated with that section. This may be quite erroneous when the tributary has an extremely mild slope which extends for many miles upstream. It may be roughly approximated that the backwater effects of the routed river flows will propagate up the tributary at the celerity of small disturbances, i.e.,

$$\hat{c} = \sqrt{g\,\bar{D}_t} \quad (142)$$

in which $\hat{c}$ is the celerity (ft/sec), g is the gravity acceleration constant, and $\bar{D}_t$ is the average hydraulic depth (ft) along the tributary within the backwater reach $(\hat{c}\Delta t)$ of the tributary. The actual reach of tributary used in Eq. (141) to determine Sa_k should not greatly exceed the backwater reach $(\hat{c}\Delta t)$. If it does, then too much of the routed river flow is stored within the tributary.

Another type of off-channel storage is a ponding area located along the river where water is stored therein but is not conveyed downstream along with the flow in the river. The connection between the ponding area and the river may be either a short conveyance channel or a broad-crested weir (sill). If the connecting channel/sill is rather narrow, then off-channel storage should be determined via Eq. (141); however, if the connecting channel/sill is very wide, then the BSS_k values may be determined by direct measurement of the storage pond widths in the direction perpendicular to the river.

When a floodplain is separated from the river by a levee that is parallel to the river, the portion of the floodplain below the crest elevation of the levee may be approximated as off-channel storage. The volume (acre-ft) within a Δx reach along the river is designated herein as V_f and the levee crest elevation as h_ℓ (ft). The off-channel storage width (BSS_k) may be computed for an elevation $(h_\ell + \hat{d}/2)$ by the following relation:

$$BSS_k = 2\ (43560)\ V_f/(\hat{d}\Delta x) \quad (143)$$

in which $\hat{d}$ is an estimated differential elevation (ft) approximated as the necessary rise in the water surface elevation above the levee crest (h_ℓ) during which the volume V_f is filled by flow leaving the river via the broad-crested weir which has an average depth above the levee crest of $\hat{d}/2$ during the interval of time for filling the V_f volume. The BSS_k values associated with the elevations h_ℓ and $h_\ell + \hat{d}$ are specified as zero. The estimation of $\hat{d}$ is not simple; for best results, it requires an iterative application of the DAMBRK model.

3.23 Selection of Manning n

Selection of the Manning n should reflect the influence of bank and bed materials, channel obstructions, irregularity of the river banks, and especially vegetation. The latter may cause the n values to vary considerably with flow elevation, i.e., the n value may be considerably larger for flow inundating the floodplain than for flow confined within the channel bank. This is due to the presence of field crops, weeds, brush, scattered trees, or thick woods located in the floodplain. Also, the n value may be larger for small floodplain depths than for larger depths. This can be due to a flattening of the brush, thick weeds, or tall grass as the flow depths and velocities increase. This effect may be reversed in the case of thick woods where, at the greater depths, the flow impinges against the branches having leaves rather than only against the tree trunks. Seasonal influences (leaves and weeds occur in summer but not in winter) may also affect the selection of the Manning n. Basic references for selecting the Manning n may be found in Chow (1959) and Barnes (1967). Also, it is recommended that two recent reports from the USGS be considered in selecting n values, i.e., Arcement and Schneider (1984) for wooded floodplains and Jarrett (1984, 1985) for relatively steep ($0.002 \leq S_o \leq 0.040$) streams with cobble/boulder beds. Both of these also provide general methodologies quite similar to that given by Chow (1959) for selecting the n value to account for the various factors previously mentioned. Arcement and Schneider (1984) also consider the effects of urbanization of the floodplain. Another methodology which estimates the Darcy friction factor (f) for floodplain flows is described by Walton and Christenson (1980). The Darcy f is related to the Manning n as follows:

$$n = 0.0926\ f^{0.5}\ D^{0.17} \qquad (144)$$

Unfortunately, the flow observations used in developing the Manning n predictive methodologies have been confined to floods originating from rainfall/snowmelt-runoff. The much greater magnitude of a dam-break flood produces greater velocities and results in the inundation of portions of the floodplain never before inundated. The higher velocities will cause additional energy losses due to temporary flow obstructions formed by transported debris which impinge against some more permanent feature along the river such as a bridge or other man-made structure. The dam-break flood is much more capable than the lesser runoff-generated flood of creating and transporting large amounts of debris, e.g., uprooted trees, demolished houses, vehicles, etc. Therefore, the Manning n values often need to be increased in order to account for the additional energy losses associated with the dam-break flows such as those due to the temporary debris dams which form and then disintegrate when ponded water depths become too great. The extent of the debris effects, of course, is dependent on the availability and amount of debris which can be transported and the existence of man-made or natural constrictions where the debris may impinge behind and form temporary obstructions to the flow.

Since dam-break floods usually have much greater velocities, it can be important for nonuniform channels to include in the Saint-Venant equations the expansion-contraction losses via the S_e term defined by Eq. (17). The ratio of expansion-contraction losses (form losses) to the friction losses can be in the range of $0.01 < S_e/S_f < 1.0$. The larger ratio occurs for very irregular channels with relatively small n values.

The Manning n for the range of flows associated with previously observed floods may be selected via a trial-and-error calibration methodology. With observed stages and flows, preferably continuous hydrographs from a previous large flood, the DAMBRK model can be used to determine the n values as follows: (1) use the observed flow hydrograph as the upstream boundary condition and select an appropriate downstream boundary (an observed stage hydrograph at the downstream boundary could be used if available); (2) estimate the Manning n values throughout the routing reach; (3) obtain computed h and Q from the solution of the Saint-Venant equations; (4) compare the computed elevations with the observed elevations at the upstream boundary and elsewhere; (5) if the computed elevations are lower than the observed, increase the estimated n values; or if the computed elevations are higher than the observed, decrease the estimated n values; (6) repeat steps (3) and (4) until the computed and observed elevations are approximately the same. The final n values are sufficient for the range of flows used in the calibration; however, the n values for those flow elevations exceeding the observed must be estimated as previously discussed. The calibrated n values, however, provide an initial estimate from which the unknown n values may be extrapolated or ultimately approximated.

It may also be mentioned that the non-Newtonian effects associated with mud/debris flows have been proposed by Chen (1984) to be accounted for by an appropriate increase in the selected Manning n values. In this approach, S_i in Eq. (9) is assumed zero by specifying MUD=0, and the friction slope (S_f) given by Eq. (10) accounts for the non-Newtonian effects.

3.24 Solution/Methodology Options

There are 12 different options for using the DAMBRK model to simulate unsteady flows in rivers and/or reservoirs. Each option is numbered according to previous documentation, e.g., the 1984 version of DAMBRK (Fread, 1984b). Herein the options are not presented in numerical order but rather in a categorical sequence. Within each category, the options are presented in the order of their recommended application on the basis of accuracy and numerical robustness.

3.24.1 Routing specified hydrograph through reaches with no dams or bridges

Option 7 is used when the flow is entirely subcritical for all times throughout the routing reach. The following special control parameters are specified to select this option: KUI = 0, KKN = 9, MULDAM = 0, and KSUPC = 0.

Option 8 is used when the flow is entirely supercritical for all times throughout the routing reach. The control parameters are: KUI = 0, KKN = 9, MULDAM = 0, and KSUPC = 1.

If the flow is mixed subcritical/supercritical, option 7 may be used with KSUPC = 2, 3, or 4 as previously described in subsection 3.7.1.

3.24.2 Routing reach has a single dam or bridge

Option 11 is used when a hydrograph is routed through a reservoir and a downstream river/valley reach. The dam may or may not breach depending on whether the specified value for h_f is attained during the simulation. The dam is treated as an internal boundary. The routing within the reservoir may be either dynamic (Saint-Venant equations) or level pool in which case Eq. (68) is used as the upstream boundary. In the first case, the specified cross sections commence at the upstream end of the reservoir or even further upstream. In the second case, the first cross section is specified for a location just upstream of the dam, and the reservoir storage characteristics are specified via a surface area-elevation table. The second cross section is specified for the tailwater section just downstream of the dam. In either case, any backwater affecting the tailwater due to channel constrictions further downstream of the dam are properly taken into account via the Saint-Venant equations. Thus, accurate tailwater elevations enable accurate submergence corrections to be used in computing the breach, spillway, gate, and/or overtopping flow at the dam. Option 11 also is used when routing a specified hydrograph through river/valley reach which includes a single bridge. By specifying KSUPC $\geq$ 2, mixed subcritical/supercritical flows may occur anywhere within the routing reach. Option 11 is selected by specifying the following control parameters: KUI = 1, KKN = 1, MULDAM = 1, and KSUPC $\geq$ 0.

Option 1 also can be used when a hydrograph is routed through a reservoir and a downstream river/valley reach. However, this option differs substantially from option 11. In option 1, the reservoir hydraulics are governed by the level pool assumption. Reservoir flows and elevations are computed via the following hydrologic storage routing technique which is based on the assumption that reservoir outflow is a single-valued function of the reservoir water level and on the law of conservation of mass, i.e.,

$$I - Q = dS/dt \qquad (145)$$

in which I is the reservoir inflow, Q is the total reservoir outflow, and dS/dt is the time rate of change of reservoir storage volume. Eq. (145) may be expressed in finite difference form as follows:

$$(I+I')/2 - (Q+Q')/2 = \Delta S/\Delta t \qquad (146)$$

in which the prime (') superscript denotes values at the time t-Δt and the Δ approximates the differential. The term ΔS may be expressed as follows:

$$\Delta S = (A_s+A_s') (h-h')/2 \qquad (147)$$

in which A_s is the reservoir surface area coincident with the elevation (h).

Using Eqs. (32) and (44) to represent the reservoir outflow Q and Eq. (147) to represent ΔS, Eq. (146) may be transformed into the following:

$$(A_s+A_s')(h-h')/\Delta t - (I+I') + Q' + k_{sp}c_sL_s(h-h_s)^{1.5} + \sqrt{2g}\; c_gA_g(h-h_g)^{0.5} +$$

$$k_dc_dL_d(h-h_d)^{1.5} + Q_t + c_vk_s[3.1b_i(h-h_b)^{1.5} + 2.45\; Z\; (h-h_b)^{2.5}] = 0 \quad .....(148)$$

in which the terms are defined in subsection 3.3.1 Since A_s is a specified function of h and all other terms except h are known, Eq. (148) can be solved for the unknown h using Newton-Raphson iteration. After obtaining h, usually within two or three iterations, Eqs. (31) and (44) can be used to compute the total outflow (Q) at time (t). In this way the outflow hydrograph Q(t) can be developed for each time step as t goes from zero to some terminating value, (t_e), sufficiently large to allow the peak of the outflow hydrograph to propagate to the downstream boundary of the river/valley reach below the dam. In Eq. (148) the time step (Δt) is chosen sufficiently small to cause minimal numerical integration error. This value is preset in the model to $\tau/50$. During the solution of Eq. (148), the tailwater elevation is computed at each time step so that the submergence correction factors (k_{sp}, k_d, k_s) in Eq. (148) can be evaluated. The tailwater elevation is approximated from the Manning equation which assumes normal, uniform flow to occur through the tailwater section and neglects backwater effects due to downstream natural constrictions or man-made constrictions such as dams and/or bridges. The Manning equation is solved for the water surface elevation using Newton-Raphson iteration. Sometimes the iterative solution diverges to an erroneous tailwater elevation. When this occurs, it is usually associated with initial discharges through the dam which greatly exceed the bankfull capacity of the tailwater cross section. Sometimes, this divergence can be eliminated by starting with initial discharges that are less than bankfull. When the tailwater diverges to incorrect values which are equal or greater than the reservoir water elevation, the information printed-out in the "Reservoir Depletion Table" is reduced to printing only the time and outflow, the latter value obtained by multiplying 0.99 times the previous outflow. This output and the method of computing the outflow is acceptable when the tailwater elevation is associated with low flows of the recession side of the reservoir outflow hydrograph and the reservoir water elevation has lowered sufficiently to approach the tailwater elevation. However, when this is not the situation, the presence of the partial table of output is a sign that the tailwater elevation has been computed erroneously. (Another way of eliminating this computational problem is to select option 11 rather than option 1.) After the outflow hydrograph has been computed, it is then routed through the downstream river/valley via the Saint-Venant equations. The flow must always be subcritical throughout the downstream routing reach. Option 1 may be selected by specifying the following control parameters: KUI = 0, KKN = 1, MULDAM = 0, and KSUPC = 0.

Option 2 can be used in lieu of option 1 when the flow through the downstream routing reach is entirely supercritical. When option 2 is used, the neglect of backwater effects in computing the tailwater elevation is appropriate since the critical flow through the spillway and breach is not effected when the tailwater flow is supercritical. Option 2 may be selected by specifying the following control parameters: KUI = 0, KKN = 2, MULDAM = 0, and KSUPC = 1.

Option 3 can be used in lieu of option 1 or 2 when the flow through the downstream river/valley consists of a reach below the dam which is entirely supercritical throughout the simulation time and a second routing reach below the first which is entirely subcritical. The hydraulic jump, which occurs as the supercritical flow changes to subcritical, is not allowed to move either upstream or downstream as the flow changes in magnitude. This restriction results in an approximation of the actual flow in the vicinity of the transition. Option 3 may be selected by specifying the following control parameters: KUI = 0, KKN = 2, MULDAM = 0, and KSUPC is specified twice (the first time KSUPC = 1 and the second time KSUPC = 0). The cross sections are specified for the supercritical routing reach and then are specified for the subcritical routing reach.

Option 4 can be used in lieu of option 1 when it is desired to use dynamic rather then level pool (hydrologic storage) routing within the reservoir. In this option, Eq. (31) is used as the downstream boundary for the reservoir. Along with other reasons for the selection of option 11, option 4 is used when 1) the breach is specified to form almost instantaneously so as to produce a negative wave within the reservoir, and/or 2) the reservoir inflow hydrograph is significant enough to produce a positive wave progressing through the reservoir. The tailwater is computed via the Manning equation and is subject to the neglect of downstream backwater effects from natural or man-made constrictions. Option 4 is selected by specifying the following control parameters: KUI = 1, KKN = 2, MULDAM = 0, KSUPC = 0 for the reservoir and again KSUPC = 0 for the subcritical downstream routing reach.

Option 5 can be used in lieu of option 4 when the downstream routing reach is entirely supercritical; this option may be selected by specifying the control parameters as follows: KUI = 1, KKN =2, MULDAM = 0, KSUPC = 0 for the reservoir and KSUPC = 1 for the supercritical downstream routing reach.

Option 6 can be used in lieu of option 3 when dynamic rather than level pool routing within the reservoir is desired. This option may be selected by specifying the control parameters as follows: KUI =1, KKN = 3, MULDAM = 0, KSUPC = 0 for the reservoir and KSUPC = 1 for the first supercritical downstream routing reach and KSUPC = 0 for the second subcritical downstream routing reach.

3.24.3 Routing reach has multiple dams and/or Bridges

Option 12 is used when a hydrograph is routed through one or more sequentially located reservoirs and/or bridges. In fact, this option is used whenever more than one internal boundary (dam or bridge) exists along the routing reach. Each dam may or may not breach depending on whether the water elevation reaches the specified h_f value for each dam and/or bridge. Each structure is treated as an internal boundary via Eqs. (30-31). The routing upstream of the first internal boundary (if it is a dam) may be either dynamic or level pool as described for option 11. The routing through all subsequent downstream reservoirs is dynamic. Tailwater elevations always include the effects of backwater from downstream constrictions. Option 12 is selected by specifying the following control parameters: KUI = 1, KKN = 1, MULDAM = number of dams and/or bridges, and KSUPC = 0. By specifying KSUPC$\geq$2, mixed subcritical/supercritical flows may occur anywhere within the system of reservoirs and/or bridges.

3.24.4 Routing reach with multiple dams

Option 12 as previously described is the preferred method for simulating a reach which has multiple dams.

Option 9 also can be used when a hydrograph is routed through a system of sequentially located reservoirs. The flow within the most upstream reservoir is routed via the level-pool (storage) routing technique described by Eq. (148). The tailwater below each dam is computed via the Manning equation; hence, backwater effects from the next downstream reservoir are neglected. The computations proceed as a sequence of routings where the outflow from an upstream dam is dynamically routed through the next downstream reservoir and this sequence is repeated until the routing is completed for the downstream reach below the farthest downstream dam. Option 9 may be selected by specifying the control parameters as follows: KUI = 0, KKN = number of dams, MULDAM = 1, and KSUPC = 0 for each dynamic routing reach.

Option 10 also can be used for a routing reach with multiple dams. It is the same as Option 9 except the flow through the most upstream reservoir is routed with the dynamic method (Saint-Venant equations). Option 10 is selected by specifying the control parameters as follows: KUI = 1, KKN = 1 + number of dams, MULDAM = 1, and KSUPC = 0 for each dynamic routing reach.

3.25 Limitations of DAMBRK Model

The DAMBRK model is subject to limitations due to its governing equations, and also due to the uncertainty associated with some of the parameters used within the model.

The governing equations within DAMBRK for routing hydrographs (unsteady flows) are the one-dimensional Saint-Venant equations. There are some instances where the flow is more nearly two-dimensional than one-dimensional, i.e., the velocity of flow and water surface elevations vary not only in the x-direction along the river/valley but also in the transverse direction perpendicular to the x-direction. Neglecting the two-dimensional nature of the flow can be important when the flow first expands onto an extremely wide and flat floodplain after having passed through an upstream reach which severely constricts the flow. In many cases where the wide floodplain is bounded by rising topography, the significance of neglecting the transverse velocities and water surface variations is confined to a transition reach in which the flow changes from one-dimensional to two-dimensional and back to one-dimensional along the x-direction. In this case, the use of radially defined cross sections along with judicious off-channel storage widths can minimize the two-dimensional effect neglected within the transition reach. The radial cross sections appear in plan-view as concentric circles of increasing diameter in the downstream direction which is considered appropriate for radial flow expanding onto a flat plane. The cross sections become perpendicular to the x-direction for the reach downstream of the transition reach. Where the very wide, flat floodplain appears unbounded, the radial representation of the cross sections is at best only an approximation which varies from reality the farther from the constricted section and the greater the variability of the floodplain topography and friction.

The high velocity flows associated with dam-break floods can cause significant scour (degradation) of alluvial channels. This enlargement in channel cross-sectional area is neglected in DAMBRK since the equations for sediment transport, sediment continuity, dynamic bed-form friction, and channel bed armoring are not included among the governing equations. The significance of the neglected alluvial channel degradation is directly proportional to the channel/floodplain conveyance ratio, since the characteristics of most floodplains along with their much lower flow velocities cause much less degradation within the floodplain. As this ratio increases, the degradation could cause a significant lowering of the water surface elevations until the flows are well within the recession limb of the dam-break hydrograph; however, in many instances this ratio is fairly small and remains such until the dam-break flood peak has attenuated significantly at locations far downstream of the dam, and where this occurs the maximum flow velocities also have attenuated. However, narrow channels with minimal floodplains are subject to overestimation of water elevations due to significant channel degradation. The effect of alluvial fill (aggradation) associated with the recession limb of the dam-break hydrograph and that occurring in the floodplain are considered to have relatively small effects on the peak flood conditions.

The uncertainty associated with the selection of the Manning n can be quite significant for dam-break floods due to: (1) the great magnitude of the flood produces flow in portions of floodplains which were very infrequently or never before inundated; this necessitates the selection of the n value without the benefit of previous evaluations of n from measured elevation/discharges or the use of calibration techniques for determining the n values; (2) the effects of transported debris can alter the Manning n as previously discussed in subsection 3.23. Although the uncertainty of the Manning n values may be large, this effect is considerably damped or reduced during the computation of the water surface elevations. The author (1981) has derived the following relation (based on the Manning equation) between the error or uncertainty in the Manning n and the resulting flow depth, i.e.,

$$d_e/d = (n_e/n)^{b'} \quad \text{.......................................(149)}$$

$$b' = 3/(3m+5) \quad \text{.......................................(150)}$$

in which d_e is the flow depth associated with an erroneous n_e value, d is the flow depth associated with the correct n value, and m is a cross section shape factor, i.e., m = 0 for rectangular sections, m = 0.5 for parabolic, m = 1 for triangular, and $1<m<3$ for. channels with floodplains (the wider and more flat the floodplain, the greater the m value). Since for channels with wide floodplains (m = 2), the exponent b' in Eq. (150) is equal to 0.27; and from an inspection of Eq. (149) it is evident that the difference between d_e and d is substantially damped relative to the difference between n_e and n. In fact, if $n_e/n = 1.5$, then $d_e/d = 1.12$, which illustrates the degree of damping. Thus for rivers with wide floodplains the uncertainty in the Manning n value results in considerably less uncertainty in the flow depths. The propagation speed (c) of the floodwave is related to the uncertainty in the Manning n according to the following:

$$c_e/c = (n_e/n)^{2b/3-1} \quad \text{.......................................(151)}$$

in which c_e is the propagation speed associated with an erroneous n_e value. If $n_e/n = 1.5$, then $c_e/c = 0.72$, which indicates less damping than that associated with Eq. (149). Thus errors in the Manning n affect the rate of propagation more than the flow depth, but in each instance the error of the flow is not proportional to the n_e error, but rather the flow error is damped.

When the range of possible Manning n values is fairly large, it is best to perform a sensitivity test using the DAMBRK model to simulate the flow, first with the lower estimated n values and then a second time with the higher estimated n values. The resulting high water profiles computed along the river/valley for each simulation represent an envelope of possible flood peak elevations within the range of uncertainty associated with the estimated n values.

Dam-break floods with a large amount of transported debris may accumulate at constricted cross sections such as bridge openings where it acts as a temporary dam and partially or completely restricts the flow. At best the maximum magnitude of this effect i.e., the upper envelope of the flood peak elevation profile can be approximated by using the DAMBRK model to simulate the blocked constriction as a downstream dam having an estimated elevation-discharge relation approximating the gradual flow stoppage and the later rapid increase due to the release of the ponded waters when the debris dam is allowed to breach.

The uncertainty associated with the breach parameters, especially $\bar{b}$ and τ, also cause uncertainty in the flood peak elevation profile and arrival times. The best approach is to perform a sensitivity test using minimum, average, and maximum values for $\bar{b}$ and τ. As mentioned previously in sub-section 2.3, the maximum flood is usually produced by selecting the maximum probable $\bar{b}$ and minimum probable τ, whereas the minimum flood is produced by using the minimum $\bar{b}$ and maximum τ values. The differences in flood peak properties (flow, elevation, time of arrival) at each section downstream of the dam due to variations in the breach parameters reduces in magnitude or is damped as the dam-break flood propagates through the downstream river/valley.

There is uncertainty associated with volume losses incurred by the flood as it propagates downstream and inundates large floodplains where infiltration and detention storage losses may occur. Such losses are difficult to predict and are usually neglected, although they may be significant. Again, a sensitivity test may be performed using estimated minimum and maximum values for (q_m) computed via Eq. (91). The conservative approach is to neglect such losses, unless very good reasons justify their consideration, e.g., observed losses associated with previous large floods in the same floodplain.

4. DATA REQUIREMENTS

The DAMBRK model was developed so as to require data that is usually accessible to the forecaster. The input data requirements are flexible insofar as much of the data may be ignored (left blank on the input data cards or omitted altogether) when a detailed analysis of a dam-break flood inundation event is not feasible due to lack of data or insufficient data preparation time. Nonetheless, the resulting approximate analysis is more accurate and convenient to obtain than that which could be computed by most other techniques. The input data can be categorized into four groups.

The first data group consists of program control parameters. They include KUI, KKN, MULDAM, and KSUPC; the combination of values specified for these parameters determines the methodology or option used to route a specified hydrograph as described previously in subsection 3.24. Other control parameters include the following: ITEH (number of discharge values in specified hydrograph), KFLP (conveyance option for computing S_f), KSL (landslide option), MUD (mud flow option), IWF (wave front tracking option), KPRES (hydraulic radius option), METRIC (metric units option), NS (number of cross sections), NCS (number of topwidths), LQ (number of lateral flow hydrographs), KCG (time dependent gate or floodplain compartment option) and JNK,NPRT,IOUTPUT,NTT,NT which control the type and extent of printed output.

The second data group pertains to the dam (the breach, spillways, and reservoir storage volume). The breach data consists of the following parameters: τ (failure time of breach, in hours); b (final bottom width of breach); Z (side slope of breach); h_{bm} (final elevation of breach bottom); h_o (initial elevation of water in reservoir); h_f (elevation of water when breach begins to form); and h_d (elevation of top of dam). The spillway data consists of the following: h_s (elevation of uncontrolled spillway crest); c_s (coefficient of discharge of uncontrolled spillway); h_g (elevation of center of submerged gated spillway); c_g (coefficient of discharge of fixed-gated spillway); c_d (coefficient of discharge of crest of dam); and Q_t (constant or time-dependent discharge from dam). The storage parameters consist of the following: a table of surface area (A_s) in acres or volume in acre-ft and the corresponding elevations within the reservoir. The forecaster must estimate the values of τ , b, Z, h_{bm}, and h_f as discussed in Section 2. The remaining values are obtained from the physical description of the dam, spillways, and reservoir. In some cases h_s, c_s, h_g, c_g, and c_d may be ignored and Q_t used in their place.

The third group pertains to the routing of the outflow hydrograph through the reservoir and/or downstream valley. This consists of a description of the cross sections, hydraulic resistance coefficients, and expansion coefficients. The cross sections are specified by their location mileage, and tables of topwidth (active and inactive) and corresponding elevations. The active topwidths may be total widths as for a composite section, or they may be left floodplain, right floodplain, and channel widths. The topwidths can be obtained from USGS topography maps, 7 1/2' series, scale 1:24000. (Westphal and Johnson (1986) compared cross-sectional data obtained from surveys and topo maps and found no significant difference in the computed flood wave properties.) The channel widths are usually not as significant for an accurate analysis as the overbank widths (the latter are available from the topo maps). The number of cross sections used to describe the downstream valley depends on the variability of the valley widths. A minimum of two must be used. Additional cross sections are created by the model via linear interpolation between adjacent cross sections specified by the forecaster. This feature enables only a minimum of cross-sectional data to be input by the forecaster according to such criteria as data availability, variation, preparation time, etc. The number of interpolated cross sections created by the model is controlled by the parameter DXM which is input for each reach between specified cross sections. The hydraulic resistance coefficients consist of a table of Manning's n versus elevation for each reach between specified cross sections. The expansion-contraction coefficients (k_{ce}) are specified as non-zero values at specified Δx reaches where significant expansion or contraction occur. The k_{ce} parameters may be left blank for most reaches.

The fourth data group is comprised of information pertaining to special options within the DAMBRK model. If the conveyance option is selected, the left and right floodplain topwidths and Manning's n versus elevation tables are specified. If the floodplain compartment option is selected, the following is specified: the number of floodplain compartments, sequence number(s) of the cross section(s) along the river connected to the compartment, crest elevation of the levee separating the river and compartment, broad-crested weir coefficient of discharge for the levee crest, initial water elevation in the compartment, volume-elevation table for the compartment, inflow hydrograph to compartment, discharge coefficient-elevation table for levee separating adjacent compartments, number of drainage pumps and their start-up/shut-off elevations and their discharge-head table for each pump in the compartment. If lateral flows exist along the river, the sequence number of the reach in which the lateral flow enters and the time series of discharges in the lateral hydrograph are specified. If a rating curve is selected for the downstream boundary condition, a table of discharge-elevation is specified; or if an elevation time series is selected for the boundary condition, the water surface elevations and associated times of occurrence are specified. If the landslide option is selected, the sequence number of cross section where the landslide occurs, its time of duration, porosity, angle of repose, and geometric description (three positions in topwidth table and its thickness) are specified. If the time dependent gate option is selected for a dam, the gate width and gate height above the sill are specified as time series along with the times of occurrence for each gate width and gate height.

The input data is completely described in detail in Appendix A; therein, the sequential order and format specifications are delineated also.

An alphabetically arranged index of data definitions is available for convenient reference in Appendix B.

New features contained in this version of DAMBRK along with modifications to the '84 DAMBRK are listed in Appendix C.

5. MODEL TESTING

The DAMBRK model has been tested with satisfactory results on at least five historical dam-break floods to determine its ability to reconstitute observed downstream peak stages, discharges, and travel times. Those floods that have been used in the testing are: 1976 Teton Dam, 1972 Buffalo Creek coal-waste dam, 1889 Johnstown Dam, 1977 Toccoa (Kelly Barnes) Dam, and the 1977 Laurel Run Dam floods. However, only the Teton and Buffalo Creek floods will be presented herein, since Toccoa and Laurel run may be found elsewhere (Land, 1980; Wurbs, 1987). The DAMBRK model also has been tested successfully on other dam-break floods in England, China, and elsewhere. Also, it has been tested satisfactorily on a laboratory scale dam-break simulation performed in 1961 by the U.S. Corps of Engineers (Waterways Experiment Station) as reported by Basco (1987).

5.1 Teton Dam Flood

The Teton Dam, a 300 ft high earthen dam with a 3,000 ft long crest and 250,000 acre-ft of stored water, failed on June 5, 1976, killing 11 people, making 25,000 homeless, and inflicting about $400 million in damages to the downstream Teton-Snake River Valley. Data from a Geological Survey Report by Ray, et al. (1977) provided observations on the approximate development of the breach, description of the reservoir storage, downstream cross sections and estimates of Manning's n approximately every 5 miles, indirect peak discharge measurements at two sites and rating curves at two sites, flood-peak travel times, and flood-peak elevations. The inundated area was as much as 9 miles in width about 16 miles downstream of the dam.

The following breach parameters were used in DAMBRK to reconstitute the downstream flooding due to the failure of Teton Dam: $\tau = 1.43$ hrs, $b = 81$ ft, $Z = 1.04$, $h_{bm} = 0.0$, $h_f = h_d = h_o = 261.5$ ft. They were obtained from the BREACH model (Fread, 1984a, 1987a). The time of failure τ was obtained by solving Eq. (7) for τ with Q_p, $\bar{b}$, h_d computed by the BREACH model. Eq. (7) can be rearranged to compute τ as follows:

$$\tau = C\ [(3.1\ \bar{b}/Q_p)^{1/3} - 1/h_d^{0.5}] \quad \text{.................................} (145)$$

in which Q_p = 2,200,000 cfs, $\bar{b}$ = 353 ft, and C = 23.4 (1936 acres)/$\bar{b}$. Cross-sectional properties were used at 12 locations along the 60-mile reach of the Teton-Snake River Valley below the dam. Five topwidths were used to describe each cross section. The downstream valley consisted of a narrow canyon (approx. 1,000 ft. wide) for the first 5 miles and thereafter a wide valley which was inundated to a maximum width of about 9 miles. Manning's n values ranging from 0.028 to 0.047 were provided from field estimates by the Geological Survey. DXM values between cross sections were assigned values that gradually increased from 0.5 miles near the dam, to a value of 1.4 miles near the downstream boundary at the Shelly gaging station (valley mile 59.5 downstream from the dam). The reservoir surface area-elevation values were obtained from Geological Survey topographic maps. The downstream boundary was assumed to be channel flow control as represented by a loop-rating curve given by Eq. (74).

The computed outflow hydrograph is shown in Fig. 2. It has a peak value of 2,172,000 cfs, a time to peak of 1.43 hrs, and a total duration of significant outflow of about 6 hrs. This peak discharge is about 30 times greater than the flood of record at Idaho Falls. The temporal variation of the computed time-integrated outflow volume compared within 3 percent of observed as shown in Fig. 3. In Fig. 4, a comparison is presented of Teton reservoir outflow hydrographs computed via reservoir storage (level-pool) routing and reservoir dynamic routing. Since the breach of the Teton Dam formed gradually over approximately a one to two hour interval, a steep negative wave did not develop. Also, the inflow to the reservoir was insignificant. For these reasons, the reservoir surface remained essentially level during the reservoir drawdown and the dynamic routing yielded almost the same outflow hydrograph as the level-pool routing technique.

The computed peak discharge values along the 60-mile downstream valley are shown in Fig. 5 along with four observed (two by indirect measurement; two by rating curves) values at miles 2.0, 8.5, 43.0, and 59.5. The average absolute difference between the computed and observed values is 5.2 percent. Most apparent is the extreme attenuation of the peak discharge as the flood wave propagates through the valley. Two computed curves are shown in Fig. 5; one in which no losses were assumed, i.e., q_m = 0; and a second in which the

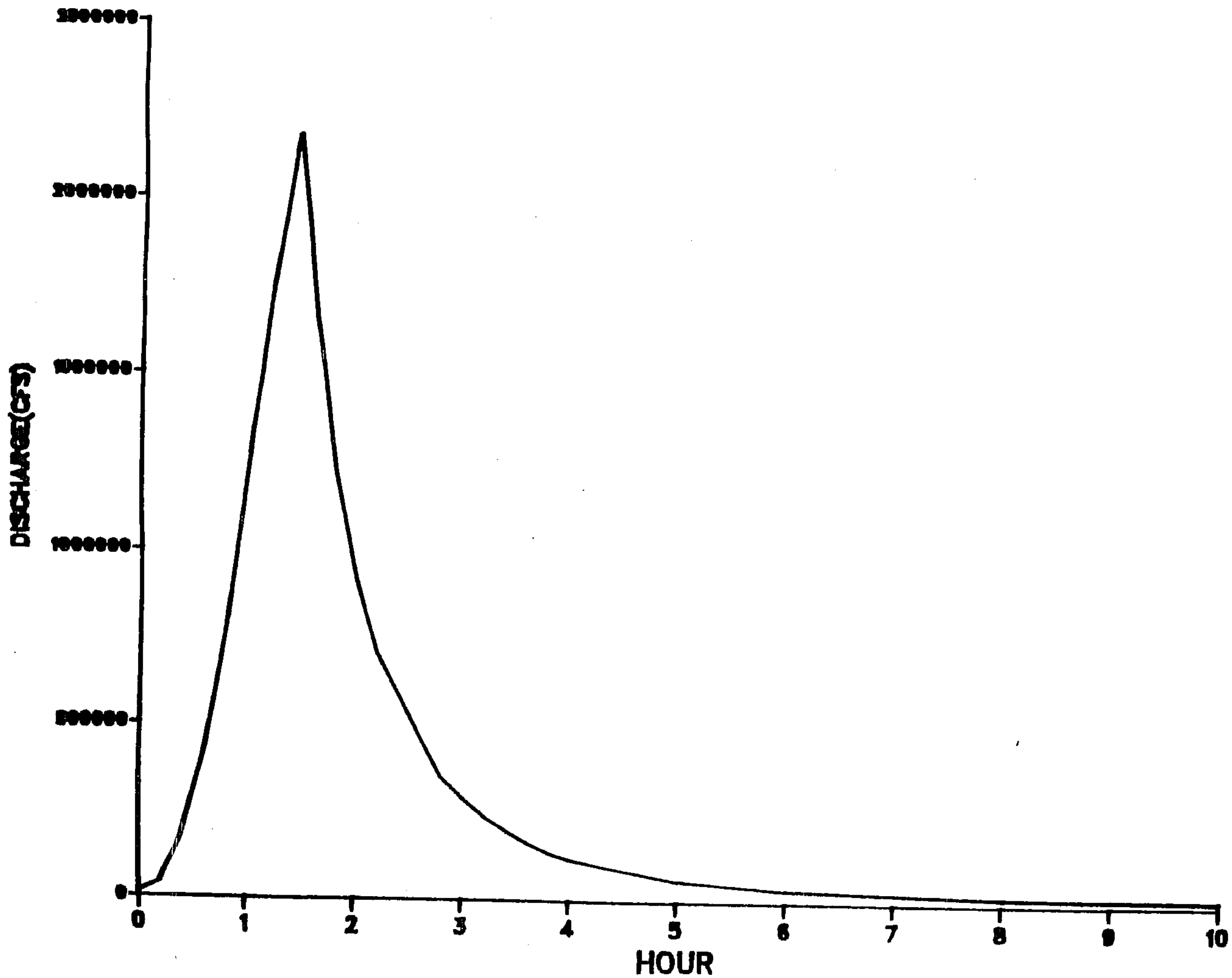


FIG. 2 — TETON BREACH OUTFLOW HYDROGRAPH

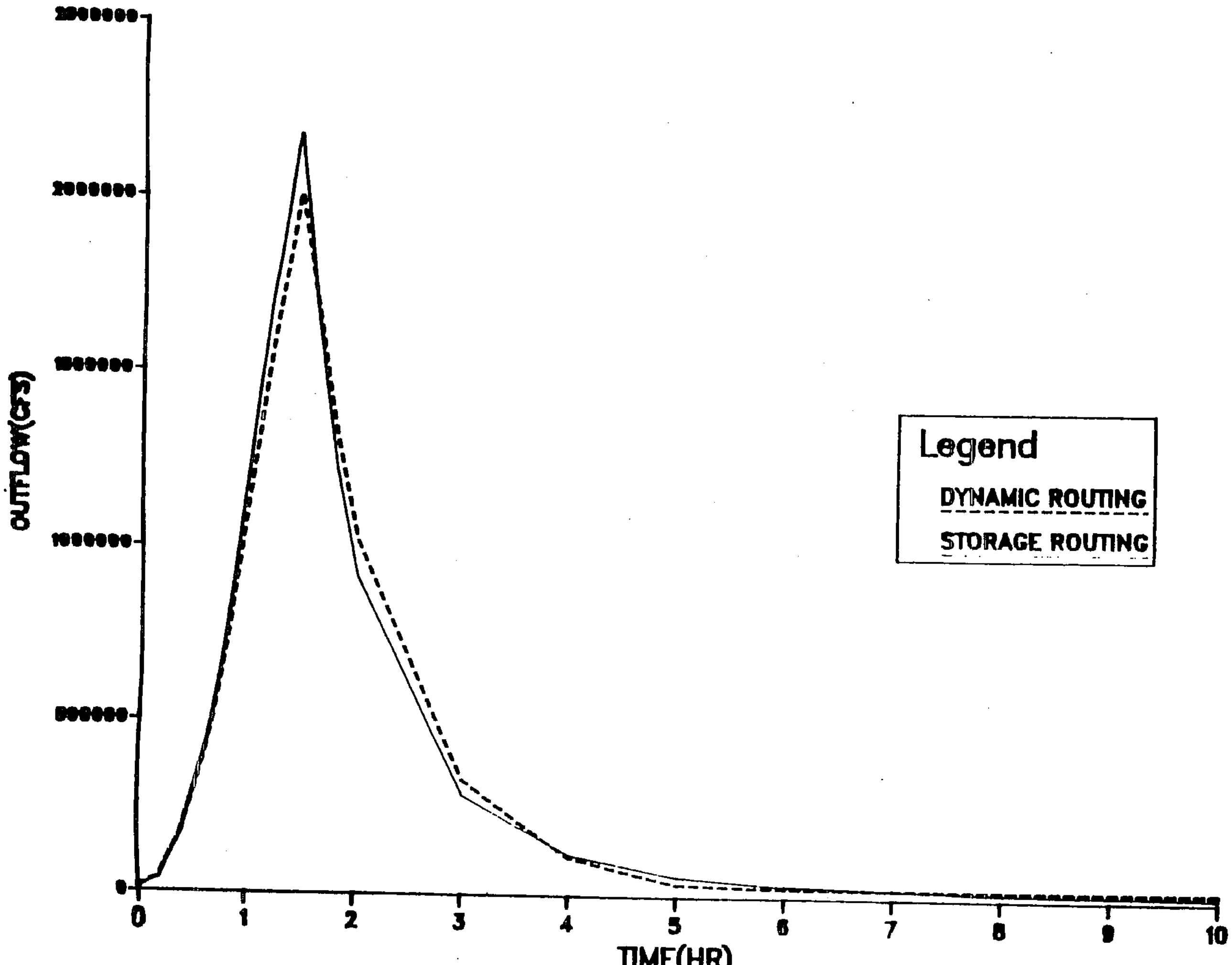


FIG. 3 – OUTFLOW VOLUME FROM TETON DAM

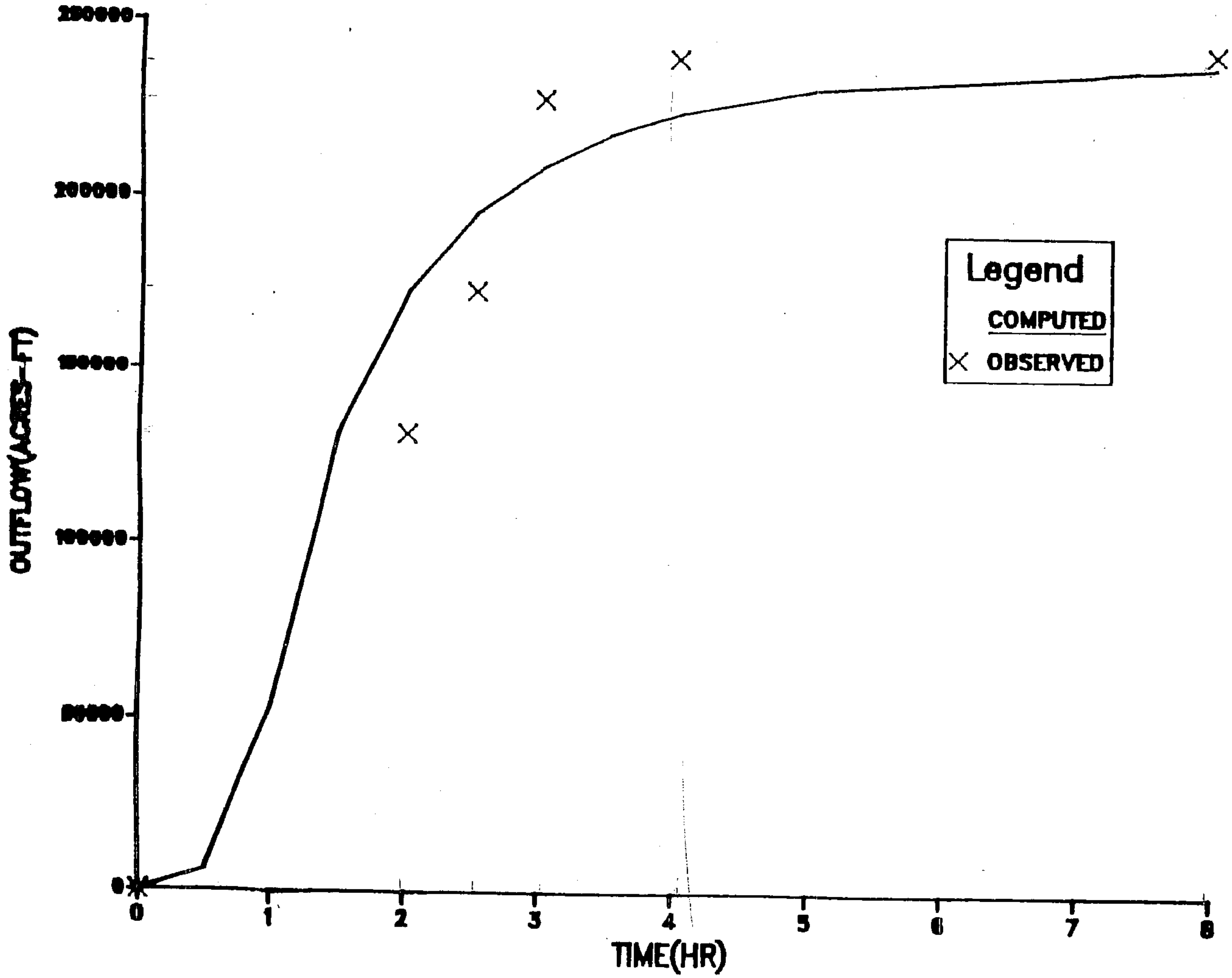


FIG. 4 – OUTFLOW HYDROGRAPH FROM TETON DAM FAILURE

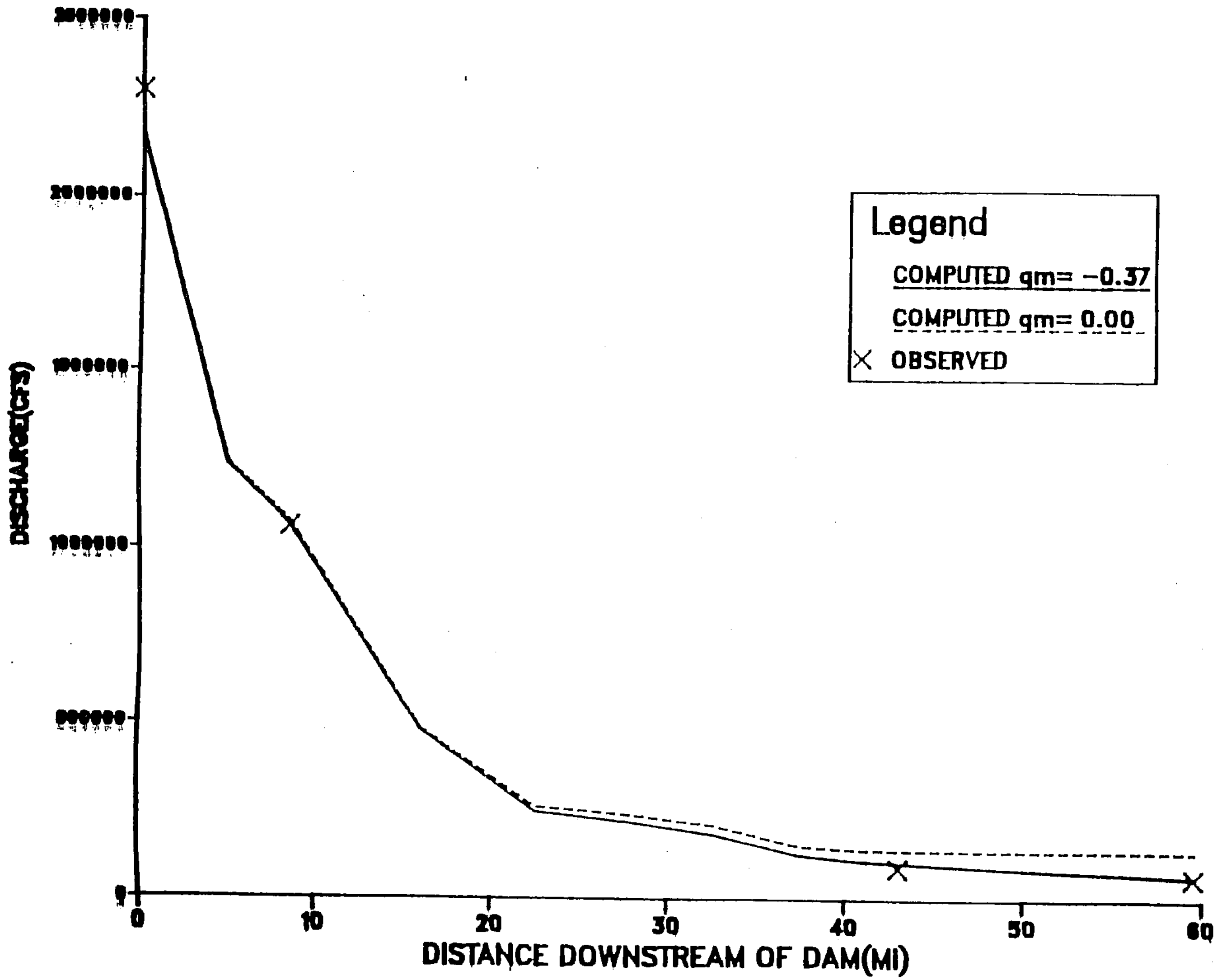


FIG. 5 – PROFILE OF PEAK DISCHARGE FROM TETON DAM FAILURE

losses were assumed to vary from zero to a maximum of q_m = -0.37 cfs/ft and were accounted for in the routing via the q term in Eqs. (8-9). Losses amounting to about 30 percent of the reservoir outflow volume were due to infiltration and detention storage behind irrigation levees. Eq. (91) was used to compute q_m.

The a priori selections of the breach parameters (τ and b) cause the greatest uncertainty in forecasting dam-break flood waves. The sensitivity of downstream peak discharges to reasonable variations in τ and b are shown in Fig. 6. Although there are large differences in the discharges (+75 to -42 percent) near the dam, these rapidly diminish in the downstream direction. After 8.5 miles the variation is about ±17 percent, and after 22 miles the variation has further diminished to about ±6 percent. The tendency for extreme peak attenuation and rapid damping of differences in the peak discharge is accentuated in the case of Teton Dam due to the presence of the very wide downstream valley. Had the narrow canyon extended all along the 60-mile reach to Shelly, the peak discharge would not have attenuated as much and the differences in peak discharges due to variations in τ and $\bar{b}$ would be more persistent. In this instance, the peak discharge would have attenuated to about 750,000 rather than 67,000 as shown in Fig. 6, and the differences in peak discharges at mile 59.5 would have been about ±17 percent as opposed to ±7 percent as shown in Fig. 6.

Computed peak elevations compared favorably with observed values, as shown in Fig. 7. The average absolute error was 1.9 ft., while the average arithmetic error was only +0.8 ft.

The computed flood-peak travel times and three observed values are shown in Fig. 8. The differences between the computed and observed travel times at mile 59.5 are about 5 percent for the case of using the estimated Manning's n values and about 13 percent if the n values are arbitrarily increased by 20 percent.

As stated previously in subsection 3.23, the Manning's n must be estimated, especially for the flows above the flood of record. The sensitivity of the computed water elevations and discharges of the Teton flood due to a substantial change (20 percent) in the Manning's n was found to be as follows: (1) 0.3 ft in computed peak water surface elevations or about 1 percent of the maximum flow depths, (2) 13 percent deviation in the computed

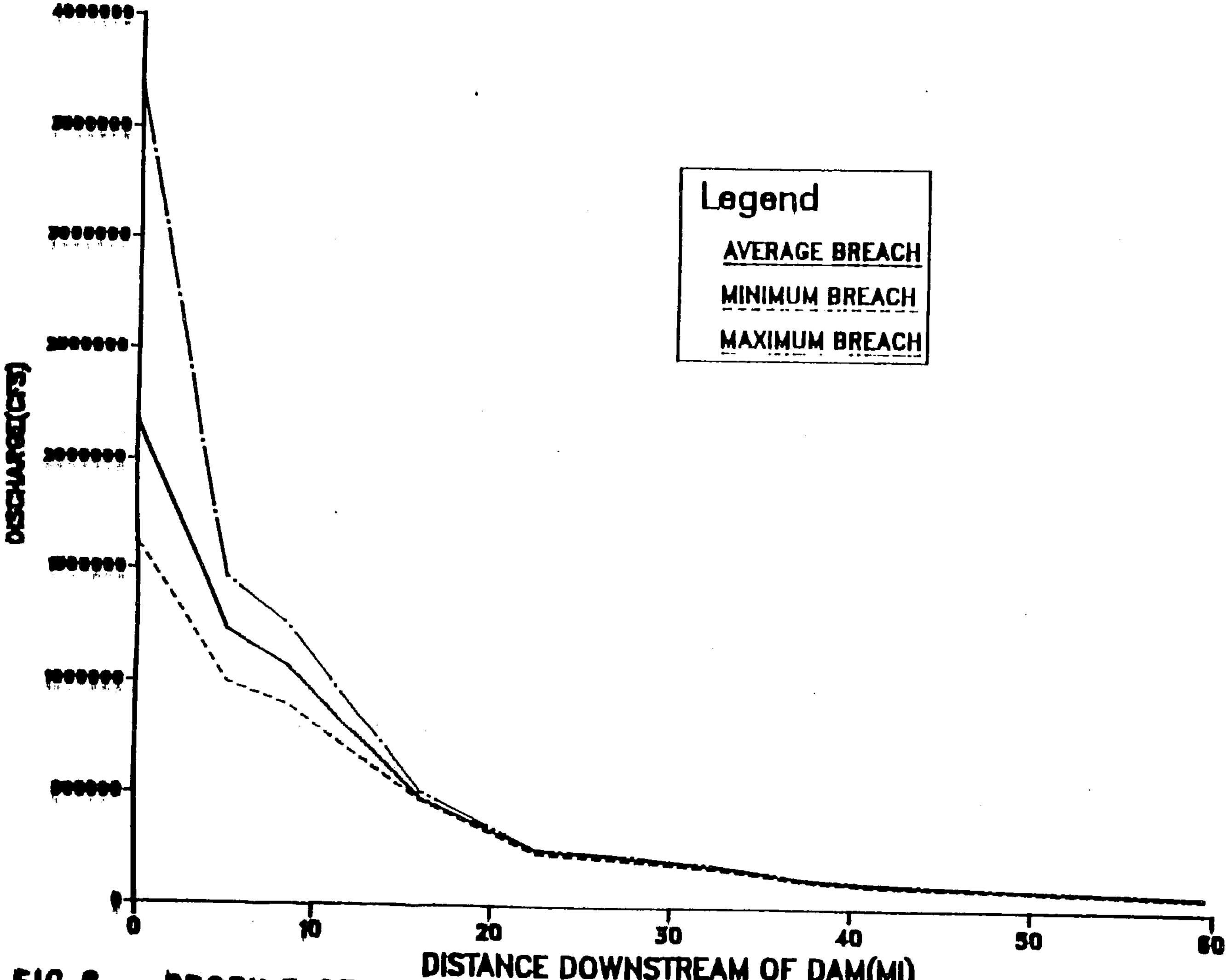


FIG 6. – PROFILE OF PEAK DISCHARGE FROM TETON DAM FAILURE SHOWING SENSITIVITY OF VARIOUS INPUT PARAMETERS

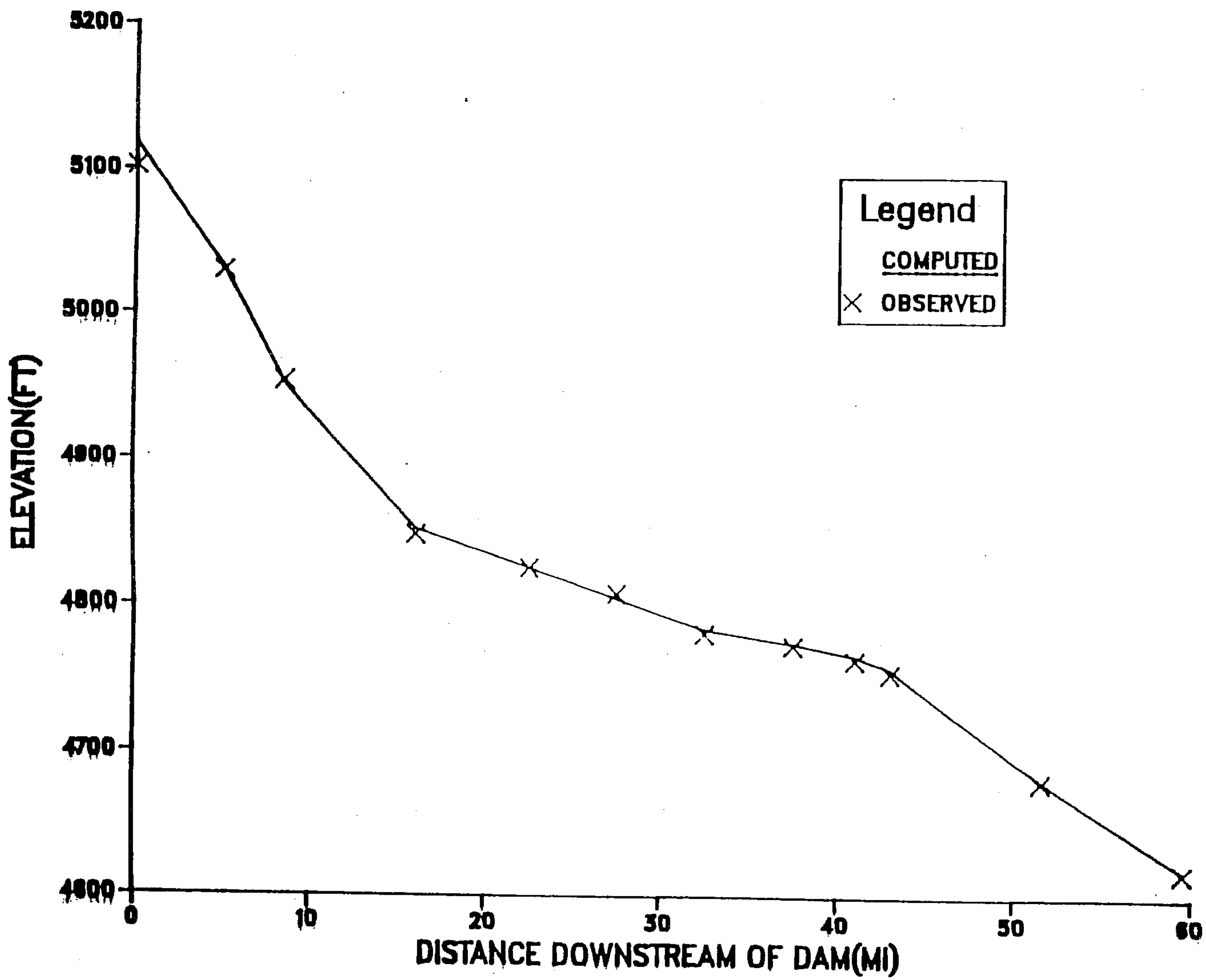


FIG. 7 – PROFILE OF PEAK FLOOD ELEVATION FROM TETON DAM FAILURE

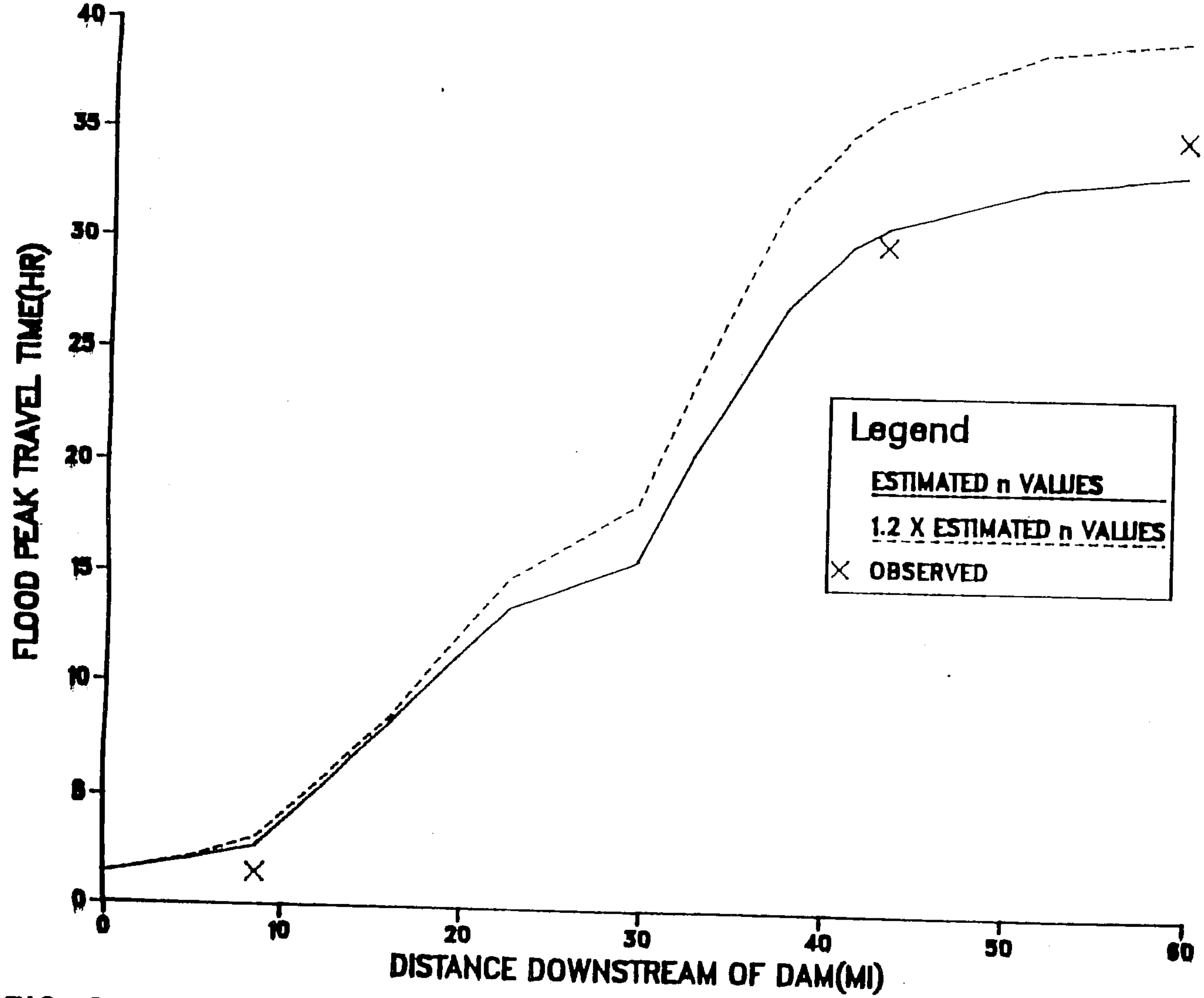


FIG. 8 – TRAVEL TIME OF FLOOD PEAK FROM TETON DAM FAILURE

peak discharges, (3) 0.5 percent change in the total attenuation of peak discharge incurred in the reach from Teton Dam to Shelly, and (4) 13 percent change in the flood-peak travel time at Shelly. These results indicate that Manning's n has little effect on peak elevations or depths; however, the travel time is affected by more than one-half of the percentage change in the n values.

A typical simulation of the Teton flood as described above involved 73 Δx reaches, 55 hrs of prototype time, and an initial time step (Δt) of 0.072 hrs which automatically increased gradually to 0.58 hrs. Such a simulation run required only 19 seconds of CPU time on an IBM 360/195 computer system. CPU time on a PR1ME 9755 was 2.7 minutes, while run time on an IBM-PC 386 compatible micro (MS-DOS 3.2, 8087 math coprocessor, 640K, MICROSOFT Fortran 4.01) was 5.7 minutes, and on an IBM-PC XT the run time was 24.5 minutes.

5.2 Buffalo Creek Flood

The DAMBRK model was also applied to the failure of the Buffalo Creek coal-waste dam which collapsed on the Middle Fork, a tributary of Buffalo Creek in southwestern West Virginia near Saunders. The dam failed very rapidly on February 26, 1972, and released about 500 acre-ft of impounded waters into Buffalo Creek valley, causing the most catastrophic flood in the state's history with the loss of 118 lives, 500 homes, and property damage exceeding $50 million. Observations were available on the approximate development sequence of the breach, the time required to empty the reservoir, indirect peak discharge measurements at four sites, approximate flood-peak travel times, and flood-peak elevations (Davies, et al., 1972). Cross sections and first estimates of the Manning roughness coefficients were taken from a report on routing dam-break floods by McQuivey and Keefer (1975).

The time of failure was estimated to be in the range of 5 minutes and the reservoir took only 15 minutes to empty according to eyewitnesses' reports. The following breach parameters were used: $\tau = 0.083$ hrs, $b = 290$ ft, $z = 0.0$, $h_{bm} = 0.0$ ft, $h_f = h_d = h_o = 44.0$ ft. Cross-sectional properties were specified for eight locations along the 15.7 mile reach from the coal-waste dam to below the community of Man at the confluence of Buffalo Creek with the Guyandotte River. The downstream valley widened from the narrow width (approximately 100 ft) of the Middle Fork to about 400-600 feet width of the Buffalo Creek valley. Minimum DXM_i values were gradually increased from 0.10 mile near the dam to 0.8 mile near Man at the downstream boundary. The reservoir area-elevation values were obtained from Davies, et al., (1972).

The 15.7 mile reach consisted of two distinct sloping reaches; one was approximately 4 miles long, with a very steep channel bottom slope (84 ft/mi), and the second extended on downstream approximately 12 miles, with an average bottom slope of 40 ft/mi. Subcritical flow prevailed throughout the routing reach for selected Manning n values of 0.060.

The reservoir storage routing option was used to generate the outflow hydrograph shown in Fig. 9. The computations indicated the reservoir was drained of its contents in approximately 15 minutes, which agreed closely with the observed emptying time. The indirect measurements of peak discharge at miles 1.1, 6.8, 12.1, and 15.7 downstream of the dam are shown in Fig. 10. The average absolute difference between the computed and observed values is 11 percent. Again, as in the Teton Dam flood, the flood peak was greatly attenuated as it advanced downstream. Whereas the Teton flood was attenuated by 78 percent in the first 16 miles of which 11 miles included the wide, flat valley below the Teton Canyon, the Buffalo Creek flood was confined to a relatively narrow valley, but was attenuated by 92 percent in the same distance. The more pronounced attenuation of the Buffalo Creek flood was due to the much more rapid breach formation and the much smaller volume of its outflow hydrograph compared with that of the Teton flood.

In Fig. 10, the computed discharges agree favorably with the observed. There are two curves of the computed peak discharge in Fig. 10; one is associated with n values of 0.06 and the other with n values of 0.090. (Comparison of computed flood travel times with the observed are shown in Fig. 11 for 0.060 n values and for the 0.090 n values.) It should be noted that the two computed curves in Fig. 10 are not significantly different, although the n values differ by a factor of 1.50. Again, as in the Teton application, the n values influence the time of travel much more than the peak discharge. The selected n values appear to be appropriate for dam-break waves in the near vicinity of the breached dam where extremely high flow velocities uproot trees and transport considerable sediment and boulders (if present), and generally result in large energy losses.

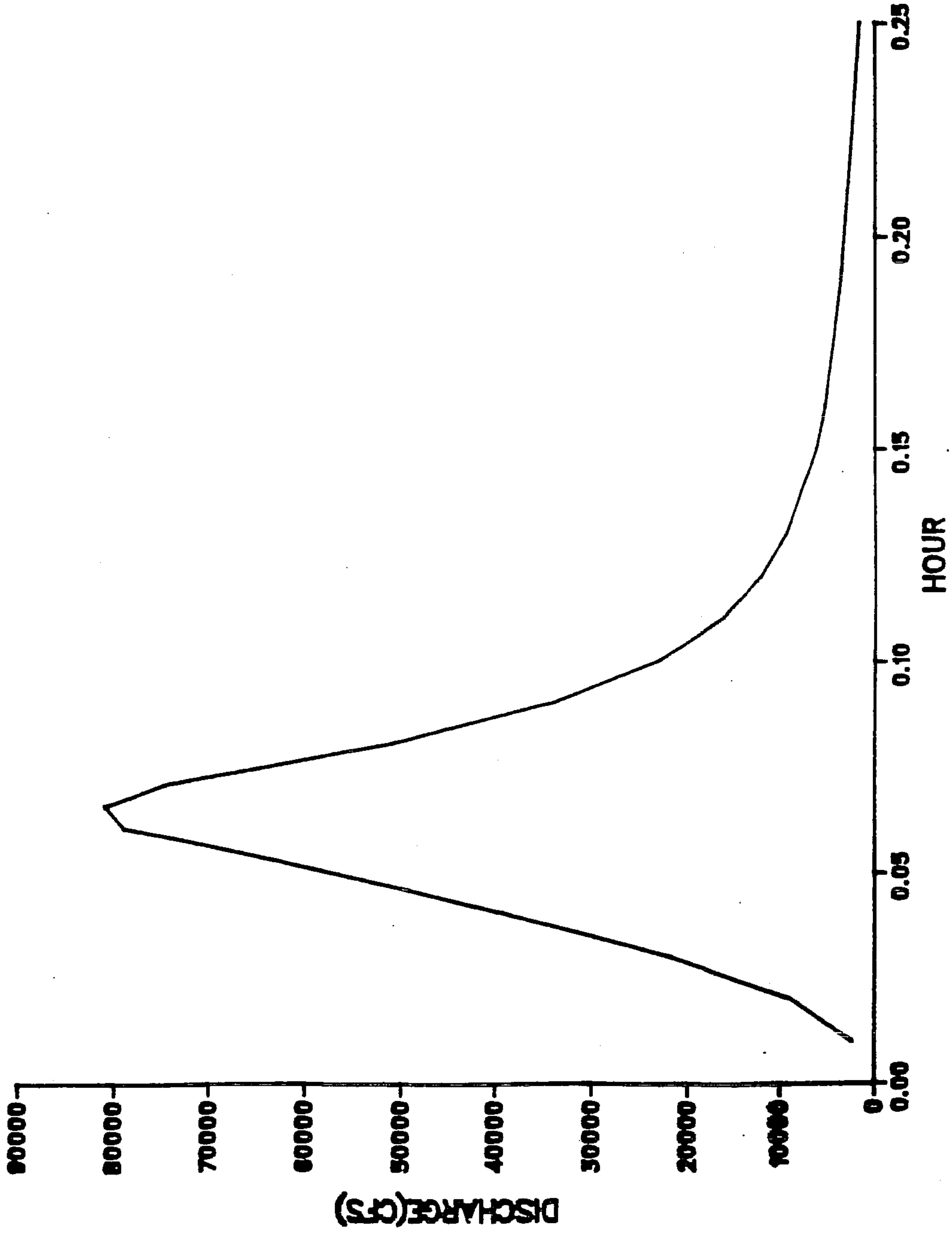


FIG. 9– BUFFALO CREEK BREACH OUTFLOW HYDROGRAPH

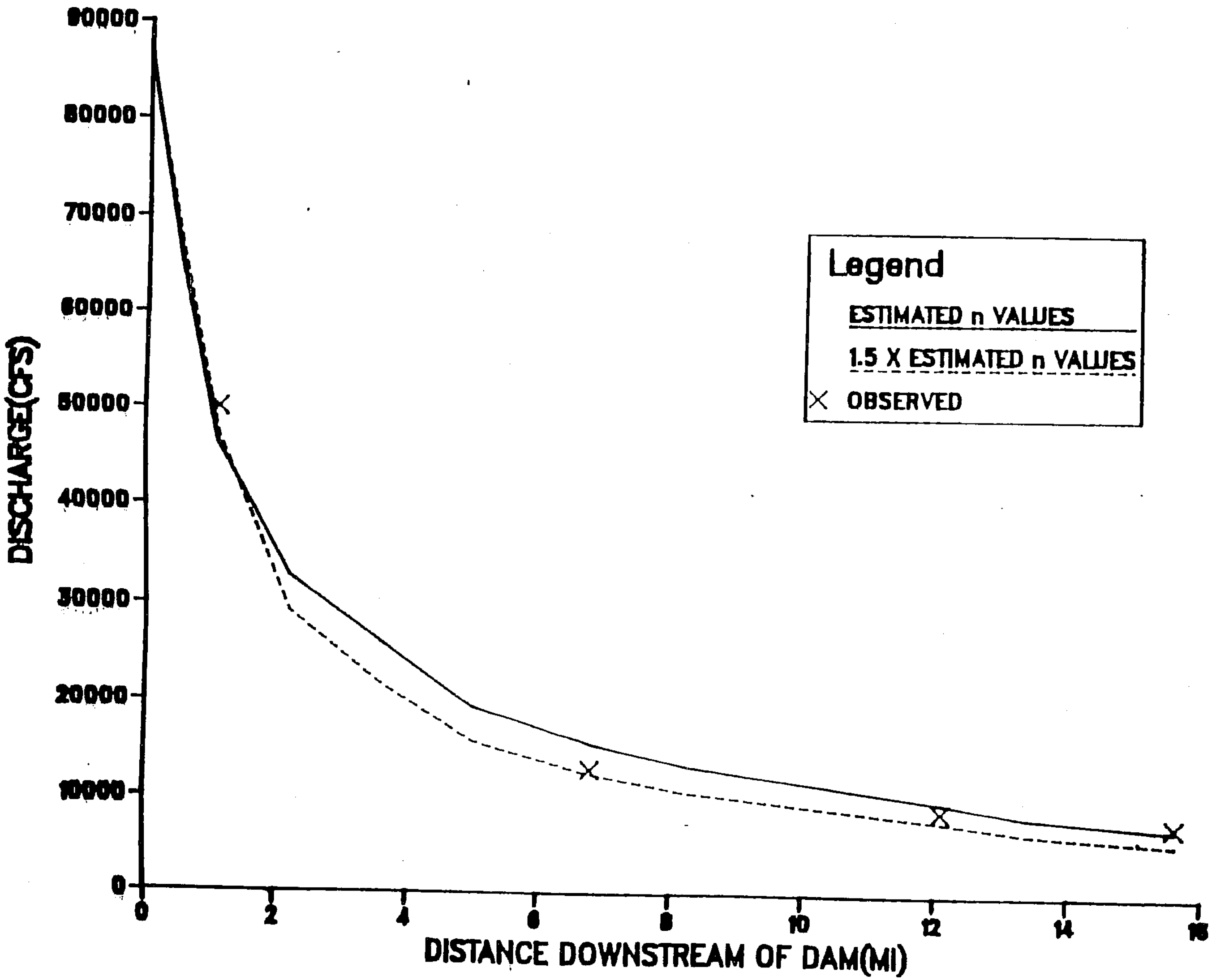


FIG. 10 – PROFILE OF PEAK DISCHARGE FROM BUFFALO CREEK FAILURE

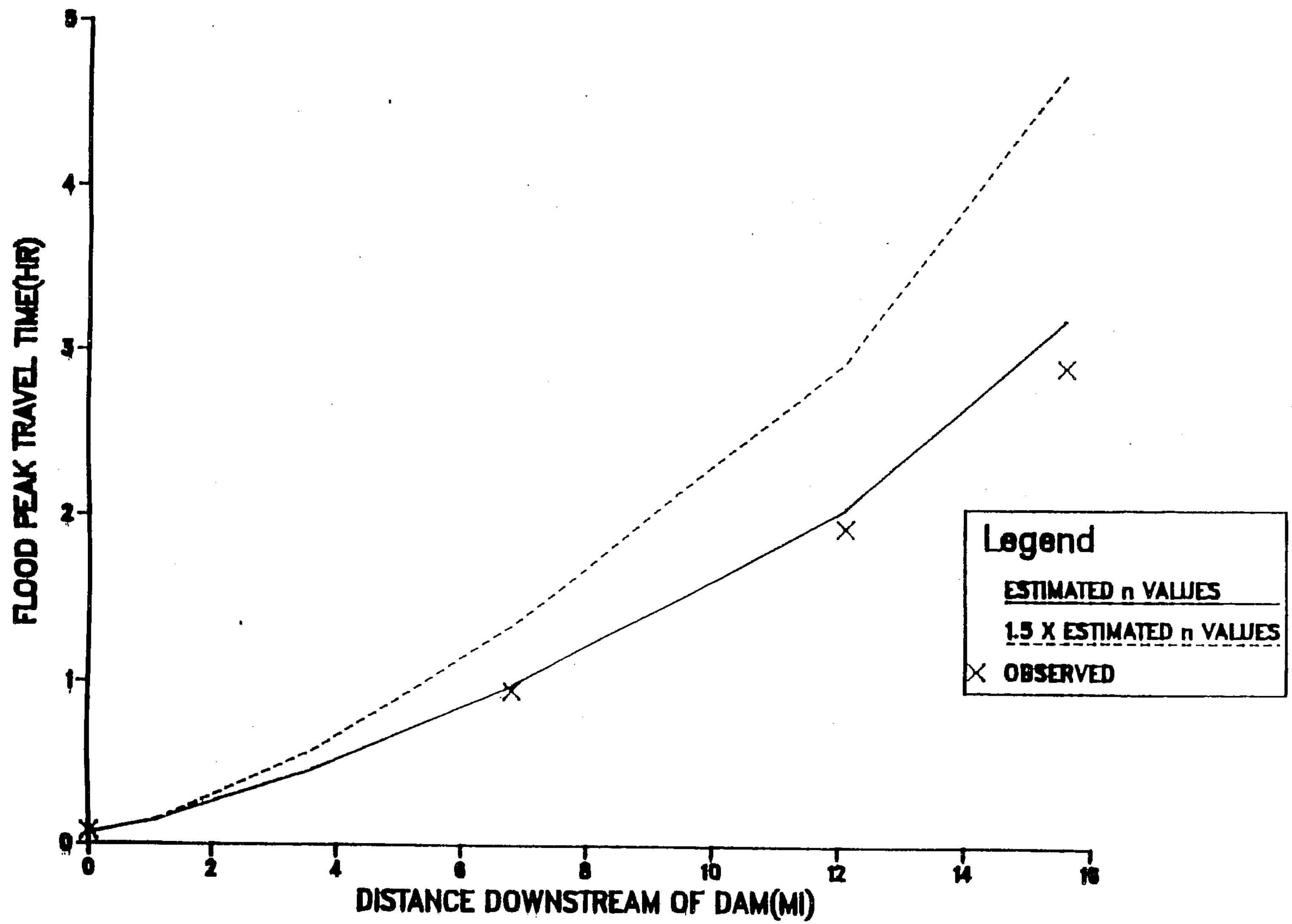


FIG. 11 – TRAVEL TIME OF FLOOD PEAK FROM BUFFALO CREEK FAILURE

A profile of the observed peak flood elevations downstream of the Buffalo Creek coal-waste dam is shown in Fig. 12, along with the computed elevations using n values increased by 50 percent. The average absolute error is 2.1 feet and the average arithmetic error is -0.9 foot.

Sensitivities of the computed downstream peak discharges to reasonable variations in the selection of breach parameters (τ, b, and Z) are shown in Fig. 13. The resulting differences in the computed discharges diminish in the downstream direction. Like the Teton dam-break flood wave, errors in forecasting the breach are damped-out as the flood advances downstream.

A typical simulation of the Buffalo Creek flood involved 198 Δx reaches, 3.0 hours of prototype time, use of the reservoir storage routing option, and time step of 0.008 hour for the subcritical downstream reach. Computation time for a typical simulation run was 75 seconds (IBM 360/195), 10.7 minutes on a PR1ME 9755, 22.5 minutes on a PC 386 compatible microcomputer, and 97 minutes on a PC XT micro.

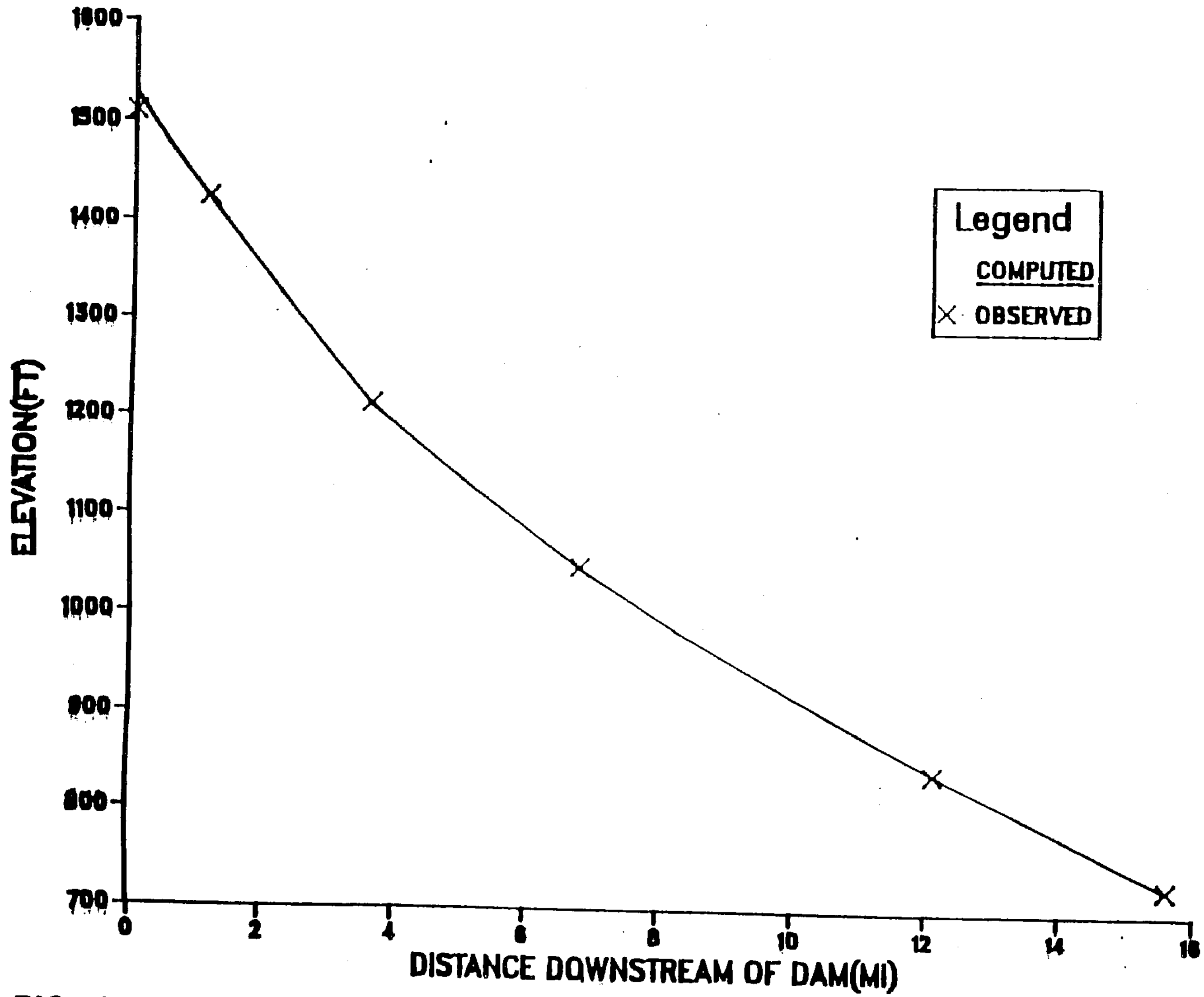


FIG. 12– PROFILE OF PEAK FLOOD ELEVATION FROM BUFFALO CREEK FAILURE

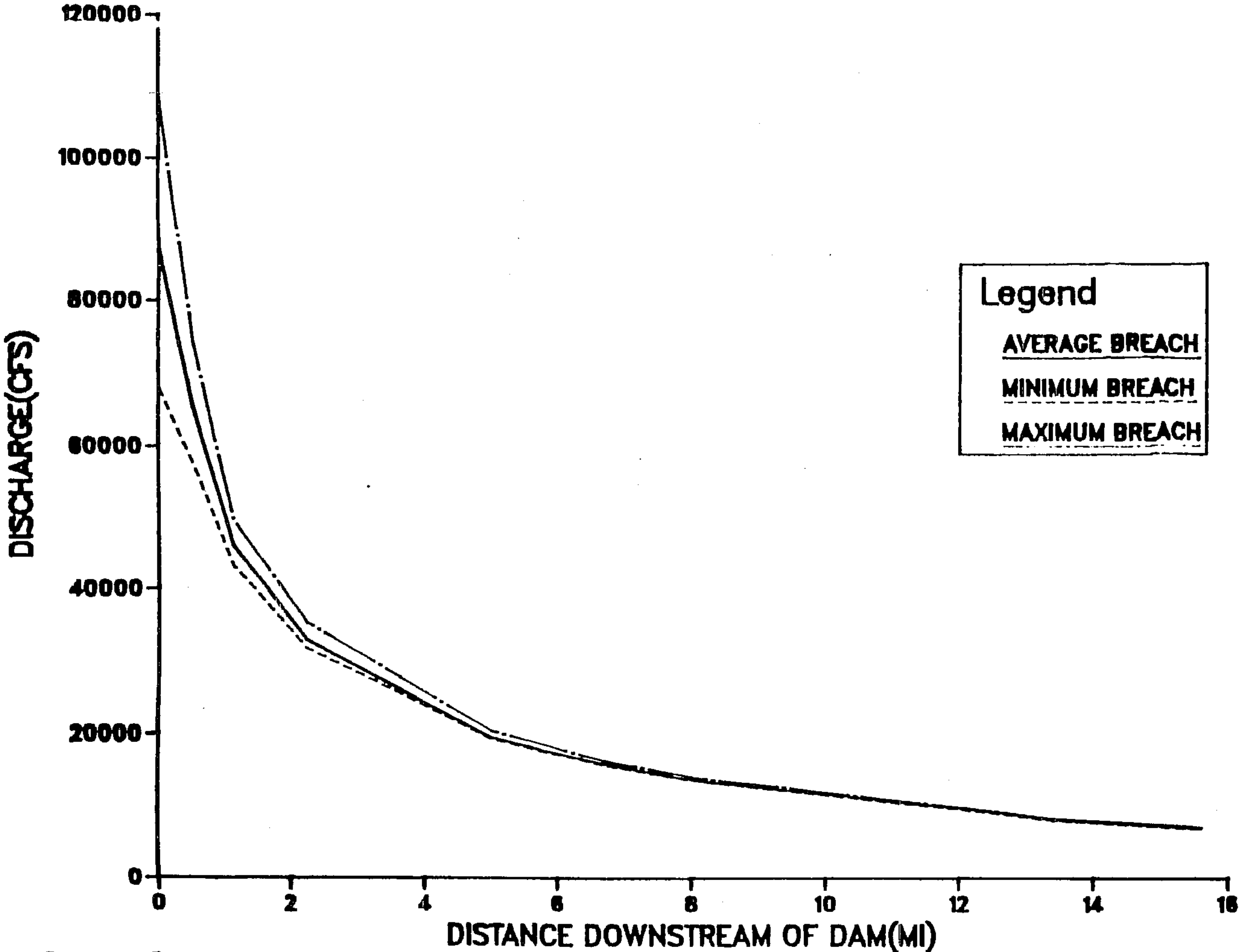


FIG. 13 – PROFILE OF PEAK DISCHARGE FROM BUFFALO CREEK FAILURE SHOWING SENSITIVITY OF VARIOUS INPUT PARAMETERS

6. FLOOD INUNDATION APPLICATIONS

The NWS DAMBRK model is suitable for the following two types of dam-break flood inundation applications: 1) pre-computation of flood peak elevations and travel times prior to a dam failure, and 2) real-time computation of the downstream flooding when a dam failure is imminent or has immediately occurred.

Pre-computations of dam failures enable the preparation of contingency flood inundation maps and/or concise graphs, and/or flash flood tables for use by those responsible for community preparedness downstream of critically located dams. The graphs provide information on flood peak elevations and travel times throughout the critical reach of the downstream valley. The variations in the precomputed values due to uncertainty in the breach parameters (τ and $\bar{b}$) can be included in the graph. Results obtained using a maximum probable estimate of $\bar{b}$ and a minimum probable estimate of τ would define the upper envelope of probable flood peak elevations and minimum travel times. Similarly, the use of a minimum probable estimated $\bar{b}$ and a maximum probable estimate of τ, would define the lower limit of the envelope of probable peak elevations and maximum travel times. In the pre-computation mode, the forecaster can use as much of the capabilities of the DAMBRK model as time and data availability warrant. The reservoir inflow hydrograph may be a "Probable Maximum Flood" (PMF) or some portion thereof. It may also be some negligible steady flow when a "sunny day" failure of the dam is simulated. Sometimes it is useful to use the DAMBRK model to simulate the progression of a PMF flood through the reservoir and downstream river/valley without allowing the dam to fail. Then, a second simulation is performed with the dam permitted to fail. The differential increase in downstream flooding is obtained by comparing the two simulations. It should be noted that the computational distance steps will usually need to be decreased in the failure simulation; this occurs since smaller time steps are needed to simulate the rapidly rising breach hydrograph, and the $\Delta x/\Delta t$ relation of Eq. (132) must be utilized to avoid numerical difficulties.

Real-time computation is also possible in certain situations where the total response time for a dam-break flood warning exceeds a few hours. An abbreviated data input to DAMBRK can be used to quickly compute an approximate crest profile and arrival times. Computer coding forms can be prepared with invariable parameters delineated and essential input data flagged. Using available topography maps and a minimum of information about the dam such as its height and storage volume, a forecast can be made in less than 30 minutes. In some cases it may be possible to make a revised forecast in real-time to update a pre-computed forecast when observations of the extent of the breach are made available to the forecaster. This would be valuable in refining the forecast for communities located far downstream where the possibility of flood inundation is questionable and the need for eventual evacuation can be more accurately defined by utilizing observations at the dam or actual flood elevations observed a few miles below the dam. The data set used to make the real-time update of the pre-computed forecast would have been retrieved from a data storage system and the critical parameters therein changed.

The DAMBRK model can also be used to route any specified flow through a river valley. In such applications of the model, the dam breach and reservoir routing data input and computational components are not used.

7. ILLUSTRATIVE EXAMPLES OF DATA INPUT

The following examples of data input illustrate some of the more frequently used options, i.e., options 1, 4, 7, 9, 11, and 12, as well as some of the special features available within the DAMBRK model, e.g., time-dependent movable-gated spillway, level-pool routing with option 11, conveyance treatment of floodplains, channel sinuosity, floodplain compartments, lateral inflows, metric input/output option, landslide-generated wave, mud/debris flow routing, mixed subcritical/supercritical routing option, and the closed conduit (pressurized flow) option. The examples include a brief physical description of the problem. A formated input data set and a listing (echo-print) of the input data set as printed-out by the DAMBRK model for each example is shown in Appendix D.

7.1 Example 1.0 -- Option 11

This example illustrates the use of option 11 to compute the outflow hydrograph from a breached dam and route it through a 59.5 mile long downstream river/valley. The routing within the reservoir and through the valley is via the dynamic (Saint-Venant) method. The dam breaches when the reservoir is overtopped by 0.5 ft. The flow is entirely subcritical. There are 11 input cross sections downstream of the dam, 2 sections at the dam, and 1 section at the upstream end of the reservoir.

The input data set is shown in Appendix D1.1. This same data set is echo-printed by the DAMBRK model with headings and definitions in tabular form is shown in Appendix D1.2.

7.2 Example 2.0 -- Option 1

This example illustrates the use of option 1 within DAMBRK to simulate the breach of a dam and route it through a 59.5 mile long downstream river/valley. Storage (level pool) routing is used within the reservoir and dynamic routing is used downstream of the dam. The flow is subcritical. There are 12 input cross sections downstream of the dam, including the tailwater section which is the first section.

The input data set is shown in Appendix D2.1. The echo-printed input data set obtained from the DAMBRK model with headings and definitions in tabular form is shown in Appendix D2.2.

7.3 Example 3.0 -- Option 12: DAM and Bridge

This example illustrates the use of option 12 to simulate unsteady flow through two structures; the first is a dam which is breached and the second is a bridge located 10 miles downstream of the dam. Level-pool routing is used for the reservoir with the tailwater elevations computed via the Saint-Venant equations. The dam and the bridge are each treated as an internal boundary. A short Δx reach bounded by cross sections at miles 0.00 and 0.01 serves as the internal boundary for the dam, and a short Δx reach bounded by cross sections at miles 10.0 and 10.01 defines the second internal boundary for the bridge. The dam breaches immediately since the initial water surface elevation of the reservoir is 1050.00 which is the same as the h_f value at which breaching commences.

The input data set is shown in Appendix D3.1. The echo-print of the same data is shown in Appendix D3.2.

7.4 Example 4.0 -- Option 11: Level pool, movable gate, conveyance

This example illustrates the use of option 11 to simulate the development of a dam-break wave due to the failure of a single dam and then dynamically route the wave through 20 miles of the downstream channel/valley. The reservoir hydraulics are treated via level-pool routing. This feature requires the input data to obey the following: (1) the parameter IDAM(1) is set to 1; (2) the first cross section is located immediately upstream of the dam (in fact, the properties of this section may be chosen to be identical with those of the tailwater section); and (3) the reservoir surface area-elevation table must be included in the input even though, for normal option 11 applications, this table is omitted. The use of time-dependent moveable gates is also illustrated in this example. This requires the parameter (KCG) to be non-zero (in this case it was set to a value of 6 indicating the number of points in the time series for the gate width and height of opening. Another special feature, the conveyance option, for treating the channel/floodplain is illustrated in this example.

The input data set is shown in Appendix D4.1. The echo-print of the same data is shown in Appendix D4.2.

7.5 Example 5.0 -- Option 7: Subcritical/supercritical

This example illustrates the use of the subcritical/supercritical mixed flow algorithm for dynamic routing of specified hydrograph through a channel reach. Some reaches are mild sloping and tend to be subcritical (-5 ft/mi $\leq$ $S_o \leq$ 5 ft/mi) which others are supercritical reaches (S_o = 20 ft/mi). The parameter (KSUPC) is set to a value of 2 which activates the mixed-flow algorithm and allows the hydraulic jump to move. The parameter (KKN) is set to a value of 9; this, along with KUI=0 and MULDAM=0 allow the option 7 to be activated. The printed-output control parameter (JNK) is set to a value of 5 which provides more information than JNK=4, particularly when the mixed-flow algorithm (KSUP$\geq$2) is used.

The input data set is shown in Appendix D5.1. This same data set is echo-printed by the DAMBRK model with headings and definitions in tabular form is shown in Appendix D5.2.

7.6 Example 6.0 -- Option 11: Free-surface/pressurized flow, lateral inflow

This example illustrates the use of free surface/pressurized flow option. The reach of channel between mile 10.1 and mile 14.9 is a closed conduit 200 ft wide and 10 ft high. The closed conduit sections have topwidths (fictious chimney width) at elevations 960.1 and 990.00 of 0.01 ft. This value for the fictitious chimney width is computed from the following: $b^* = gA/\hat{C}^2$ in which A = 2000 ft^2 and $\hat{C}$ = 2538 ft/sec.

The input data set is shown in Appendix D6.1. This same data set is echo-printed by the DAMBRK model with headings and definitions in tabular form is shown in Appendix D6.2.

7.7 Example 7.0 -- Option 7: Floodplain compartments

This example illustrates the use of the floodplain compartment option. This particular problem is concerned with routing a specified hydrograph through a 2.5 mile long rectangular channel with subcritical flows. Therefore, option 7 is utilized, i.e., KKN=9, KUI=0, MULDAM=0, and KSUPC=0. There are two floodplain compartments on each side of the river; they are located between mi. 1.0 and 1.5 and between mi. 1.5 and 2.0. The average levee crest elevation of the two most upstream compartments is 105.00 and 104.50, respectively, for the left and right compartments. The levee crest elevations of the two downstream compartments are 104.00 and 103.00, respectively, for the left and right compartments.

The input data is shown in Appendix D7.1. The same data set is echo-printed by the DAMBRK model with headings and definitions in tabular form as shown in Appendix D7.2.

7.8 Example 8.0 -- Same as example 2.0 except with metric option

This example illustrates the metric option which is activated by specifying the input parameter (METRIC) equal to 1. The equivalent English/metric units for input used in DAMBRK is shown in Table 1. This example is identical to example 2.0, an option 1 type of simulation. In the metric option both the input and output are in metric units rather than English units.

The input data set is shown in Appendix D8.1. This same data set is echo-printed from DAMBRK and shown in Appendix D8.2.

7.9 Example 9.0 -- Option 5: Supercritical flow downstream of dam

This example illustrates the use of option 5 which simulates unsteady flow and the development of a breach hydrograph at a dam via dynamic routing. Tailwater elevations, which are used to determine tailwater submergence effects, are computed via the Manning equation applied to the tailwater section. The outflow hydrograph is then routed through a 26 mile reach of channel/valley downstream of the dam. The flow in the downstream reach is always supercritical; hence, KSUPC is entered with a value of 1.

The input data set is shown in Appendix D9.1. This same data set is echo-printed from DAMBRK and shown in Appendix D9.2.

7.10 Example 10.0 -- Option 9: 2 Dams

This example illustrates the use of Option 9 to simulate the breach of an upstream dam immediately at t=0 since the initial water surface elevation (yo)

Table 1. English/Metric Equivalents in DAMBRK

Property	English Unit	Metric Unit	Conversion Factor (English to Metric)
Time	hr	hr	
Length	ft	m	1/3.281
Length	mile	km	1.6093
Flow	ft^3/sec	m^3/sec	1/35.32
Area	ft^2	m^2	1/10.765
Surface Area	acres	km^2	1/247.1
Volume	acre-ft	$10^6\ m^3$	1/810.833
Weir Coeff.	$ft^{1/2}/sec$	$m^{1/2}/sec$	1/1.811
Unit Weight	lb/ft^3	N/m^3	157.1
Shear Strength	lb/ft^2	N/m^2	47.88
Viscosity (Dynamic)	$lb\ sec/ft^2$	$N\ sec/m^2$	47.88
Manning n	English and Metric are same		1.0

is 5288.5 which is the same as the elevation required for breaching which is specified as HF = 5288.5. The breach hydrograph is obtained from the failure of the upstream dam using level-pool routing for the upstream reservoir. The breach hydrograph is then dynamically routed through a downstream reach in which the last 20 miles is a reservoir contracted by a second dam located 59.5 miles downstream of the first dam. The second dam fails when it is overtopped by at least 0.5 ft. This hydrograph, consisting of the superposition of the breach hydrograph of the second dam onto the routed breach hydrograph from the upstream dam, is then routed through a 60-mile reach of channel.

The input data set is shown in Appendix D10.1. The same data set is echo-printed by the DAMBRK model and shown in Appendix D10.2.

7.11 Example 11.0 -- Option 7: Mudflow

This example illustrates the use of the mudflow option to dynamically route a specified mudflow hydrograph through a 1.0-mile reach of channel in which the flow changes from subcritical to supercritical. Therefore, option 7 is utilized, i.e., KKN=9, KUI=0, MULDAM=0, KSUPC=4, AND MUD=3. The mudflow has an apparent viscosity of 10 lb.sec/ft^2, unit weight of 125 lb/ft^3, initial shear stress of 20 lb/ft^2, and the non-Newtonian behavior is that of a Bingham plastic requiring POWR=1.0. The channel has an initial steady flow of 50 ft^3/sec.

The input data is shown in Appendix D11.1. The same data set is echo-printed by the DAMBRK model with headings and definitions in tabular form as shown in Appendix D11.2.

7.12 Example 12.0 -- Option 4: Landslide wave

The example illustrates the use of a model option which generates a wave via a landslide into a reservoir. The reservoir is 5.0 miles in length; the landslide occurs between miles 3.4 and 3.9. The thickness of the landslide (maximum depth of slide measured perpendicular to the longitudinal axis of the reservoir) varies from 0.0, 50., 100., 50., and 0.0 ft. at miles 3.4, 3.5, 3.65, 3.8, and 3.9, respectively. The lowest vertical point of the landslide occurs at the 4th elevation in the topwidth table at each cross-section. The highest vertical point occurs at the 6th elevation of the topwidth table. The landslide rushes into the reservoir in 36 seconds. An option 4 simulation methodology is used; thus, dynamic routing is used in the reservoir to create the landslide-generated wave which produces a 4-ft. high wave at the dam. This causes the already full reservoir to be overtopped, but no failure of the dam is assumed to occur. The overtopping flow is then routed through a 1 mile reach of the downstream valley.

The input data set is shown in Appendix D12.1. This same data set is echo-printed by the DAMBRK model with headings and definitions in tabular form as shown in Appendix D12.2.

8. MODEL OUTPUT

The DAMBRK model output is controlled by the parameters JNK and IOPUT. The JNK control parameter is a general control whereas IOPUT controls specific output. JNK may be assigned values of 1, 4, 5, 9, 10, and 12 where the output becomes more extensive as JNK increases. It is recommended that for most runs, JNK be specified as 4; this output is considered to provide a maximum of information for the least number of pages of output. A JNK=1 provides the least output and is intended to be used for obtaining final results to minimize permanent paper or file storage requirements. JNK values > 4 are to be used to obtain detailed hydraulic and numerical information for confronting and overcoming numerical difficulties that have caused aborted runs or suspect results.

The output consists of the following types:

- Type D: Echo-Print of Data Input described in Appendix D
- Type E: Bottom Slope Profile Table as described in Appendix E
- Type F: Reservoir Depletion Table described in Appendix F
- Type G: Initial Condition Table described in Appendix G
- Type H: Initial Water Elevations and Bottom Slope Profile Plot as described in Appendix H
- Type I: Minimal Dynamic Routing Information at Each Time Step described in Appendix I
- Type J: Nonconvergence Information for Dynamic Routing at Each Time Step described in Appendix J
- Type K: Subcritical/Supercritical Reach Information as described in Appendix K
- Type L: Maximum Amount of Dynamic Routing Information (Hydraulic Information at Each or Selected Cross Sections) at Each Time Step described in Appendix L
- Type M: Type L + Dynamic Routing Information at each Iteration as described in Appendix M
- Type N: Type M + Normal Critical, and Sequent Elevation Information at each Iteration as described in Appendix N

Type O: Plot of Maximum Discharge Profile Information as described in Appendix O

Type P: Crest Profile Table Containing Maximum Flows, Elevations, and their Times of Occurrence at Each Computational Section as described in Appendix P

Type Q: Hydrograph Plots as described in Appendix Q

Type R: Computed Elevation and Discharge Tables for plotting as described in Appendix R

Type S: Internal Boundary Information at Each Time Step (Append. S)

Type T: Type L Output with Floodplain Compartment Option (Append. T)

Type U: Crest Profile Table with Conveyance (Floodplain) Option (Append. U)

Type V: Type L Output with Conveyance (Floodplain) Option (Append. V)

A JNK=1 provides types D, F, and P; types O and Q are also provided if they are permitted via the parameters IOPUT(7) and NTT, respectively. The cross section portion of TYPE D is controlled by IOPUT(9).

A JNK=4 provides types D, F, and P; types E, G, H, I, O, and Q are also provided if they are permitted via the parameters IOPUT (1), IOPUT(10), IOPUT(3,4), IOPUT(5), IOPUT(7), and NTT=0, respectively.

A JNK=5 is identical to that of JNK=4 except types J and K are also provided.

A JNK=9 is identical to that of JNK=5 except type E is omitted while types L and R are also provided where type R is controlled by IOPUT(8).

A JNK=10 is identical to that of JNK=9 except type M is also provided.

A JNK=12 is identical to that of JNK=10 except type N is also provided.

A summary presentation of various types of output and the manner in which each may be controlled is shown in Table 2.

Generally, output variables are defined categorically with the first or first two letters; i.e., Q is discharge, Y is water surface elevation, X is cross section distance location, FR is Froude number, T is time, V is velocity, A is wetted cross-sectional area, B is wetted cross-sectional topwidth, and CM is the Manning n. Further definitions of output variables may be found in Appendices D through V.

Table 2. DAMBRK MODEL OUTPUT CONTROL

Output Description	Type	JNK						IOPUT( ) CONTROL
		1	4	5	9	10	12	
Input Echo-Print	D	x	x	x	x	x	x	9
Bottom Slope Table	E	x	x	x	x	x	x	
Reservoir Depletion Table	F		x	x	x	x	x	10
Initial Condition Table	G		x	x	x	x	x	3,4
Initial Water Elevations & Bottom Slope Plot	H		x	x	x	x	x	1
Min Dynamic Routing Info	I		x	x	x	x	x	5
Nonconvergence Info	J			x	x	x	x	
Subcritical/Supercritical Info	K			x	x	x	x	
Dynamic Routing Info	L				x	x	x	
L + Dynamic Routing Info at Each Iteration	M					x	x	
M + Normal and Sequent Depth Iteration Info	N						x	
Crest Profile Plots	O	x	x	x	x	x	x	7
Crest Profile Table	P	x	x	x	x	x	x	
Hydrograph Plots	Q	x	x	x	x	x	x	NTT≥0
Hydrograph Tables	R				x	x	x	8
Internal Boundary Info	S		x	x	x	x	x	
Floodplain Compartment Info	T			x	x	x	x	
Crest Profile Table for Floodplain	U	x	x	x	x	x	x	

9. MODEL PROGRAM STRUCTURE

The DAMBRK model is programmed in Fortran and may be used with compilers which accept either Fortran '77 or Fortran '66 standards. The model is disseminated in both a source form and an executable (compiled) form.

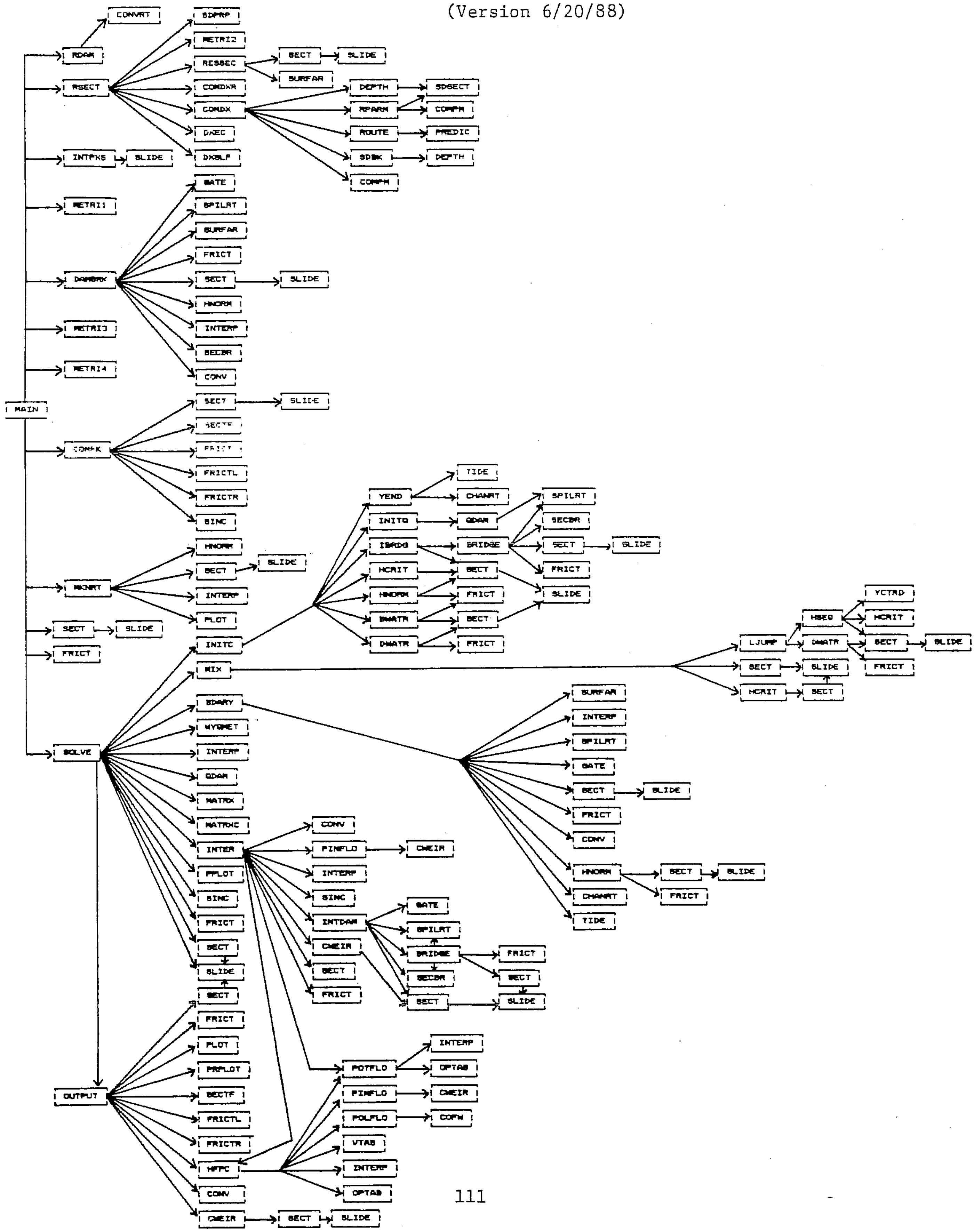
The DAMBRK model is modular in structure; it consists of 72 subroutines. It requires 640K storage to execute on a computer. A definition of the principal function of each subroutine is shown in Appendix V. The subroutines are interconnected as shown in Fig. 14.

9.1 Enlargement of Program for Simulations that Exceed Design Specifications

The DAMBRK model has fixed dimensions for all arrays used in the subroutines. The arrays are found in COMMON and DIMENSION statements located at the beginning of each subroutine. A categorical definition of the size of the arrays is given as follows:

number of input cross sections..........................	90
number of input and interpolated cross sections........	200
number of inflow or lateral flow hydrograph ordinates...	100
number of topwidths for a cross section................	8
number of computational time steps......................	700
number of hydrograph plots.............................	6
number of internal boundaries (structures).............	10
number of lateral inflow/outflow locations.............	12
number of floodplain compartments......................	20
number of pumps in all floodplain compartments.........	25
number of subcritical/supercritical sub-reaches........	21
number of conveyance points............................	30
number of smoothing reaches............................	7
number of interpolated landslide sections..............	31
number of selected sections for detailed simulation output...	32

FIG. 14 - DAMBRK Flow Chart
(Version 6/20/88)



The various arrays may be enlarged beyond the above dimensions if a computer having storage capacity in excess of 640K is used to run the DAMBRK model. A knowledge of the variables and their definition is not required in order to change the array sizes. For example, if the total number of cross sections are increased from 200 to 600 the following steps are required:

1. In every subroutine, in every COMMON and DIMENSION statement having an array, e.g., DXM(200), the number 200 is changed to the number 600.

2. In every subroutine, in every COMMON and DIMENSION statement having an array, e.g., C(400), the number 400 (twice 200) is changed to the number 1200 or two times the value of 600.

3. In subroutine MAIN, the executable statement KXMAX = 200 is changed to 600.

Another example might be to change the number of time steps from 700 to 1000. This would involve the same first step described in the first example except the number 700 is changed to the number 1000. The second step is not required and the third step, in this case, would involve changing the statement KSTP = 699 to KSTP = 999.

Yet another example might be to change the number of internal boundaries from 10 to 14. The first step would be as in the first example except the number 10 is changed to the number 14. The second and third steps are not required for this example, nor for any of the other possible arrays defined by the remaining categories delineated previously.

Of course, any subroutine in which a change is made would have to be compiled and a new executable (load) module created before the program could be run with the enlarged array size.

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INDEX

APPENDIX A -- Input Data Structure for DAMBRK Model: Version 1988-4

Input card group no.

*(1) MDAM, MRVR, MNAME - 20 A 4 Format

MDAM	Name of dam (col. 1-20).
MRVR	Name of reservoir (col. 21-40).
MNAME	Agency name (col. 41-60).

MESAGE - 20 A 4 Format

MESAGE	Agency address--street, room (col. 1-40). Agency Address--city, state, zip code (col. 41-72).

*(2) KKN, KUI, MULDAM, KDMP, ITEH, NPRT, KFLP, METRIC - 8 I 10 Format

KKN	Parameter to control options 1-12. (See Page A-21)
KUI	Parameter to control options 1-12. "
MULDAM	Parameter to control options 1-12. "
KDMP	Parameter for printing; users outside of the National Weather Service set KDMP=3 or 5. If KDMP=5, IOPUT on card (4) allowing selective printout of computations is read-in and KDMP is reset to 3.
ITEH	Parameter denoting number of hydrograph ordinates of inflow hydrograph to reservoir; maximum value of 100 is allowed; if ITEH=0, the inflow hydrograph is generated via a mathematical Gamma function.
NPRT	Parameter to control print output for JNK=9, NPRT is the total number of cross sections (maximum of 32) at which hydraulic information is printed-out during dynamic routing; if NPRT=0, the program uses a variable NPRT computed by the program and prints-out hydraulic information at NPRT intervals of cross sections along the routing reach.
KFLP	Parameter denoting the use of conveyance for computing the friction slope (S_f); if KFLP=0, the conveyance feature is not used; if KFLP=1, the conveyance feature is used.
METRIC	Parameter indicating if input/output is English or metric units; 0 is English; 1 is metric.

* Input data card group required for any simulation.

Note: Card (3) is omitted unless NPRT (card (2)) >0.

(3) NPT(K) - 8 I 10 Format

NPT(K) Sequential number of cross section after interpolation at which hydraulic information is printed-out; K index goes from 1 to NPRT where NPRT $\leq$ 32.

Note: Card (4) is omitted unless KDMP (card (2)) = 5.

(4) IOPUT(K) - 10 I 1, 2 I 2 Format

IOPUT(K) Optional print parameter that may override the JNK parameter, (card 20). K index goes from 1 to 12. If IOPUT(K)=0, allow the output to be printed; if IOPUT(K)=1, suppress the output. The following output can be controlled:

Col	
1	Slope profile plot
2	Summary tables of input cross-sections and reaches
3	Initial conditions table (reversed)
4	Initial conditions table - backwater elevation table (forward)
5	Dynamic routing - at upstream and downstream boundaries
6	Dynamic routing - at each multiple dam site (similar to depletion table)
7	Summary plots - peak elevation, discharge, time to peak, and time to flood elevation
8	Arrays for selected hydrograph plots
9	List of input cross-sectional information
10	Reservoir depletion table
11-12	This value represents the time (integer hours) at which printing of output will commence; all output will be suppressed until this time is reached.
13-14	The interval at which the output will be printed.

Note: Card (5) is omitted unless KKN = 1, KUI = 1, MULDAM $\geq$ 1 (Card (2)).

(5) IDAM(K) - 8 I 10 Format

IDAM(K) — Number of cross section coincident with the upstream face of each dam; K index goes from 1 to MULDAM. This parameter is only read-in when the simultaneous computation of the complete system is desired (see note on page A-23 for further information on the use of this computational option). The option to create cross sections within the most upstream reservoir from surface area (SA) - elevation (HSA) table is activated by specifying IDAM(1) as a negative number whose absolute value also represents the total number of cross sections to be automatically created within the upstream reservoir.

Note: Cards (6) and (7) are omitted if KUI=1 except as described in Note on page A-24 concerning Level-Pool Routing for options 11 & 12.

(6) SA(K) - 8 F 10.0 Format

SA(K) — Surface area (acres) or volume (acre-ft) of reservoir at elevation HSA(K). Maximum of 8 values allowed. Area or volume determined by VOL parameter (card (8)). Surface area is preferred since reservoir storage (level-pool) routing uses area, and the volume to area conversion algorithm does not always provide a good transformation.

(7) HSA(K) - 8 F 10.0 Format

HSA(K) — Elevation (ft) at which reservoir surface area SA(K) is defined; elevation is referenced to a datum plane corresponding to mean sea level (m.s.l.). Elevations start at highest and proceed to lowest. Maximum of 8 values allowed. Lowest elevation must be $\leq$ YBMIN as defined on card (8).

Note: Cards (8) and (9) are omitted if KKN = 9.

(8) RLM, YO, Z, YBMIN, BB, TFH, DATUM, VOL - 8 F 10.0 Format

RLM — Length (mi) of reservoir; only used when option to create reservoir cross sections is used (see card (5)).

YO — Elevation (ft) of water surface in reservoir when computation commences; elevation is referenced to m.s.l. datum. When card (8) represents a bridge (see note on page A-23), YO = 0.0.

Z — Side slope (1:vertical to z:horizontal) of breach.

YBMIN — Lowest elevation (ft) that bottom of breach reaches; elevation is referenced to m.s.l. datum.

BB — Width (ft) of base of breach.

TFH — Time (hr) from beginning of breach formation until it reached its maximum size.

DATUM — Elevation (m.s.l.) of bottom of dam.

VOL — Parameter indicating if SA(K) is surface area (acres) or volume (acre-ft); if VOL=0.0, SA(K) is acres; if VOL=1.0, SA(K) is acre-ft and SA(K), K = 1,8 will be automatically converted to acres; if VOL > 1.0 where VOL = SA(1), in acres, SA(K) is acre-ft and SA(K), K = 2,8 will be automatically converted to acres. The last option (VOL > 1.0) helps provide a reasonable conversion from volume to surface area and should be used if at all possible. Of course, if a surface area table is known, it should be used rather than a volume table.

(9) HF, HD, HSP, HGT, CS, CG, CDO, QT - 8 F 10.0 Format

HF — Elevation (ft) of water when failure of dam commences; elevation is referenced to m.s.l. datum; if HF is entered with a negative value, failure will commence at a certain time given by |HF| where HF has units of hours.

HD — Elevation (ft) of top of dam; elevation is referenced to m.s.l. datum; if HD is entered as a negative value, the length of the dam crest is variable with elevation as described for cards (10 & 11).

HSP — Elevation (ft) of uncontrolled spillway crest; elevation is referenced to m.s.l. datum; if HSP is entered as a negative value, the failure starts in the spillway at its crest and failure is confined to a length along the dam of CS/3.0 which approximates the length of the spillway.

HGT — Elevation (ft) of center of gate openings; elevation is referenced to m.s.l. datum. Also, elevation (m.s.l.) of bottom of sill of time-dependent gate.

CS — Discharge coefficient for uncontrolled spillway; it is equal to the coefficient of discharge (2.6-3.2) times the length (ft) of the spillway.

CG — Discharge coefficient for gate flow; it is equal to the coefficient of discharge (0.60-0.80) times the area of gates (ft^2) divided by 8.025. (If the last change shown on MAIN.F77 is 9/19/89 or later, do not divide by 8.025).

CDO — Discharge coefficient for uncontrolled weir flow over the top of the dam; it is equal to the coefficient of discharge (2.6-3.2) times the length of the dam crest (ft) less the length of the uncontrolled spillway and gates; if HD is negative, CDO is entered as the average discharge coefficient (2.6-3.2). If CDO is entered as a negative value, the breach will be via piping with the initial center elevation (ft) of the pipe at $|-CDO|$.

QT   Discharge (cfs) through turbines; this flow is assumed constant from start of computations until the dam is ¼ breached; thereafter, QT is assumed to linearly decrease to zero when ½ breached. QT may also be considered leaking or constant spillway flow. If this flow is time-dependent, QT is entered with any negative value and the time series for QT is specified via card (12).

NOTE: Cards (10-11) are always read-in for bridges, i.e., YO = 0.0; however for dams (YO>0.0) they are omitted unless HD is specified with a negative sign.

(10) HSBR(K,L) - 8 F 10.0 Format

HSBR(K,L)   Elevation (ft. m.s.l.) associated with widths of bridge/culvert opening. Start at invert and proceed upwards. K goes from 1 to NCS. Also, for dams, elevations associated with the variable length of the dam crest. L index goes from 1 to MULDAM (card (2)) which may be maximum of 10; if MULDAM = 0, L goes from 1 to 1.

(11) BSBR(K,L) - 8 F 10.0 Format

BSBR(K,L)   Width (ft) associated with bridge/culvert opening corresponding to each elevation, HSBR(K,L). Also, for dams, the variable length of dam crest for a given elevation; L and K indices are the same as described for card (10).

Note: Card (12) is omitted unless QT (card (9)) is negative.

(12) QTT(K,L) - 8 F 10.0 Format

QTT(K,L)   Variable discharge (cfs) through turbines; this flow is time dependent; K index goes from 1 to ITEH which can assume a maximum value of 50; L index is same as described for card (10).

Note: Cards (13) and (14) are read-in only if either HSP is non-zero and CS is zero, or HGT is non-zero and CG is zero. This option allows a rating curve to be used for either the uncontrolled spillway or submerged gate rather than an equation for each using a constant discharge coefficient as in Eq. (32).

(13) QSPILL(K,L) - 8 F 10.0 Format

QSPILL(K,L) Flow (cfs) of spillway or gate rating curve; K goes from 1 to maximum of 8; L goes from 1 to MULDAM (card (2)) which may be a maximum of 10; if MULDAM=0, L goes from 1 to 1.

(14) HEAD(K,L) - 8 F 10.0 Format

HEAD(K,L) Head (ft) above spillway crest or gate center; head is associated with spillway flow or gate flow in rating curve; K goes from 1 to maximum of 8; L goes from 1 to MULDAM.

Note: Repeat cards (10-14) as L index goes from 1 to MULDAM. If MULDAM=0, L index goes from 1 to 1.

*(15) DHF, TEH, BREX, MUD, IWF, KPRES, KSL - 3 F 10.0, 4I10 Format

DHF Interval (hr) between QI(K) input hydrograph ordinates; enter 0.0 if intervals are not equal.

TEH Time (hrs) from beginning of routing until routing is terminated.

BREX Exponent used in development of breach; varies from 1. to 4.; zero default is 1.0.

MUD Parameter denoting mud/debris flow; 0 or -1 is dynamic routing of non-mudflow (water); -2 is Muskingum-Cunge routing of non-mudflow; 2 is Muskingum-Cunge routing of mudflow.

IWF Parameter indicating dry bed routing; 0 indicates use of base flow at t=0 all along the routing reach; 1 indicates wave front tracking where $V_W = V_{N-4}$, if IWF = 2, $V_W = K_W V_{N-4}$; and if IWF = 3, $V_W = V_{max_i}$, i = 1,2,...N, in which V_W is the wave front velocity, N is the current location of the wave front, and K_W is the kinematic wave factor (Eq. (113) in paper).

KPRES Parameter indicating method of computing hydraulic radius; 0 indicates R=A/B, 1 indicates R=A/P where P is wetted perimeter.

KSL — Landslide parameter; if KSL=0, no landslide; if KSL=1, a landslide along one bank of the reservoir is simulated; if KSL=2, the landslide occurs along both banks of reservoir; landslide on opposite bank is mirror image of the one that is specified on cards (55-56).

DFR — Window for critical Froude No. in mixed flow algorithm. $0.0 < DFR \leq 0.30$; default to 0.05.

Note: Card (16) is omitted if MUD=0 (card (15)).

(16) UW, VIS, SHR, POWR - 4 F 10.0 Format

UW — Unit weight (lb/ft^3) of mud/debris fluid
VIS — Dynamic viscosity (lb • sec/ft^2) of mud/debris fluid
SHR — Initial yield stress or shear strength (lb/ft^2) of mud/debris fluid.
POWR — Exponent in power function representing the stress-rate of strain relation, if Bingham plastic is assumed for fluid, set POWR = 1.0.

Note: Omit card (17) if ITEH (card 2) is nonzero. If card (17) is included, then omit cards (18) and (19).

(17) QO, RHO, GAMA, TPG - 4 F 10.0 Format

QO — Initial steady discharge (cfs).
RHO — Ratio of peak flow to initial flow of inflow hydrograph for Gamma mathematical function to create inflow hydrograph.
GAMA — Ratio of time between t=0 and center of gravity of inflow hydrograph to time between t=0 and hydrograph peak.
TPG — Time (hr) between t=0 and inflow hydrograph peak.

*(18) QI(K) - 8 F 10.0 Format

QI(K) — Inflow (cfs) at upstream end of reservoir or first routing reach for each interval of time until time TEH is reached; K goes from 1 to ITEH which can assume a maximum value of 100; omit if ITEH=0.

Note: Card (19) is omitted if DHF>0.0 (card (15)) or ITEH=0 (card (2)).

(19) TI(K) - 8 F 10.0 Format

TI(K) Time associated with QI(K); if TEH exceeds TI(ITEH), then QI(ITEH+1) = QI(ITEH) and TI(ITEH+1) = TEH are automatically set, K goes from 1 to ITEH.

*(20) NS, NCS, NTT, JNK, KSA, KSUPC, LQ, KCG - 8 I 10 Format

NS Number of cross sections used to describe the channel and valley downstream of dam; first cross section should be immediately downstream of dam; last cross section should be at farthest point downstream of dam where flood information is desired; other cross sections can be located as desired by user; maximum of 90 and minimum of 2 cross sections can be used to describe the downstream channel valley. Also, if dynamic routing is used in reservoir, specified cross sections for reservoir are included as part of the NS sections with the first cross section located near upstream end of reservoir.

NCS Maximum number of top widths used to describe a cross section. Maximum allowable value is 8.

NTT Total number of cross sections at which discharge hydrographs will be plotted; maximum number is limited to 6. The location of the cross sections, at which plots are provided, is specified by the parameter NT(K), which is on card (21). If NTT=0, no plots are provided. If NTT=a negative value between 1 and 6, the profile plots are suppressed.

JNK Parameter to specify the type of output other than plots which will be provided; if JNK=0, a minimum of output is provided--this includes all input data and hydrograph plots; if NTT=0, no hydrographs or other output printed; if JNK=1, reservoir depletion table printed, profile of downstream crests and times, and designated hydrographs; if JNK=4, additional information is printed at each time step for debugging; if JNK≥9, considerable information is printed for debugging. For normal runs JNK = 4 is recommended since enough information is printed to allow some analysis but not excessive output. JNK can have the following values: 0, 1, 4, 5, 9, 10; the higher the number, the more information is printed-out.

KSA Parameter to indicate type of cross-section smoothing. If KSA<0, then smoothing of cross sections will be automatically performed. Type of smoothing is specified on card (22).

KSUPC Parameter to indicate if flow is supercritical. If KSUPC=0, flow through entire downstream channel-valley reach is subcritical and no special treatment is required; if KSUPC=1, the flow is known to be supercritical throughout the entire downstream reach. If the flow can vary from supercritical to subcritical or subcritical to supercritical with time and location along the routing reach, KSUPC = 2, 3 or 4. If KSUPC = 2, the hydraulic jumps can move upstream or downstream. If KSUPC = 3, the hydraulic jumps move only if the Froude number exceeds 2.0. (This increases the numerical robustness). If KSUPC = 4, the hydraulic jumps can never move. If the flow is nearly critical along several adjacent reaches, KSUPC = 3 or 4 should be specified since these options are more robust (numerically) and this is needed for near-critical flow.

LQ — Parameter denoting the total number of lateral inflow hydrographs along the downstream channel-valley; a maximum of 12 hydrographs, each with a maximum of 100 ordinates, are allowed.

KCG — Number of ordinates in spillway gate control curve of gate coefficient (CGCG) vs. time (TCG) described on cards (58) and (60). If KCG is negative, it activates the floodplain compartments option and represents total number of floodplain compartments (maximum of 20 allowed).

Note: Card (21) is omitted unless NTT >0, (card 20)).

(21) NT(K) - 6 I 10 Format

NT(K) — Number of cross sections (1 through NS) at which hydrograph plots are desired; K goes from 1 to NTT.

Note: Cards (22 - 23) are omitted unless KSA<0, (card (20)).

(22) SMF, NTSM, NSMR - F10.2, 2 I 10

SMF — Smoothing factor, $0.5 \leq SMF \leq 0.9$. The larger SMF values produces greater smoothing.

NTSM — Parameter indicating type of smoothing. If NTSM=1, smoothing of widths along x-axis; if NTSM=2, smoothing of widths in vertical where maximum width/ft change is $|KSA|$*50; if NTSM=3, smoothing of elevations along x-axis; NTSM=4, type 1 and type 2 smoothing; if NTSM=5, type 1, 2, and 3 smoothing.

NSMR — Number of separate smoothing reaches within the total routing reach; maximum of 7 allowed.

(23) NUSM(K), NDSM(K) - 2 I 10

NUSM(K) Upstream cross-section number of K^{th} smoothing reach.

NDSM(K) Downstream cross-section number of K^{th} smoothing reach.

Note: Card (23) is read-in for each K^{th} smoothing reach as K goes from 1 to NSMR.

*(24) XS(I), FSTG(I), - 2 F 10.0 Format

XS(I) Location (mi) of cross sections used to describe downstream channel/valley; mileage must increase in the downstream direction. If KFLP=1 (card (2)), XS(I) is mileage measured along mean flow path of floodplain.

FSTG(I) Elevation (m.s.l.) at which flooding commences; may be left blank.

*(25) HS(K,I) - 8 F 10.0 Format

HS(K,I) Elevation (ft), referenced to m.s.l. datum, corresponding to each top width (BS(K,I)) on card (26) used to describe cross section; K goes from 1 to NCS; NCS values of HS(K,I) are entered on a single card. NCS is limited to a maximum of 8. Start with lowest HS and proceed to highest value of HS.

*(26) BS(K,I) - 8 F 10.0 Format

BS(K,I) Top width (ft) of active flow portion of channel/valley cross section corresponding to each elevation HS(K,I); K goes from 1 to NCS; NCS values of BS(K,I) are entered on a single card; NCS is limited to maximum of 8.

Note: Card (27) is omitted unless KFLP = 1, (card (2)).

(27) BSL(K,I) - 8 F 10.0 Format

BSL(K,I) Top width (ft) of active flow portion of left floodplain corresponding to each elevation HS(K,I); K goes from 1 to NCS; NCS values of BSL(K,I) are entered on a single card; NCS is limited to a maximum of 8.

Note: Card 28 is omitted unless KFLP = 1, (card (2)).

(28) BSR(K,I) - 8 F 10.0 Format

BSR(K,I) Top width (ft) of active flow portion of right floodplain corresponding to each elevation HS(K,I).

*(29) BSS(K,I) - 8 F 10.0 Format

BSS(K,I) Top width (ft) of off-channel storage portion of channel/valley cross section corresponding to each elevation HS(K,I); K goes from 1 to NCS; NCS values of BSS(K,I) are entered on a single card; NCS is limited to maximum of 8.

Note: Cards (24)-(29) are repeated for each cross section as in the index I goes from 1 to NS.

*(30) CM(K,I) - 8 F 10.0 Format

CM(K,I) Manning n for channel corresponding to each elevation HS(K,I); K goes from 1 to NCS; NCS values of CM(K,I) are entered on a single card; NCS is limited to maximum of 8; the Manning n represents the roughness enountered by the flow through the reach bounded by cross sections at locations I and I+1.

Note: Card (31) is omitted unless KFLP = 1, (card (2)).

(31) CML(K,I) - 8 F 10.0 Format

CML(K,I) Manning n for left floodplain corresponding to each elevation HS(K,I); K goes from 1 to NCS; NCS values of CM(K,I) are entered on a single card; NCS is limited to a maximum of 8.

Note: Card (32) is omitted unless KFLP = 1, (Card (2)).

(32) CMR(K,I) - 8 F 10.0 Format

CMR(K,I) Manning n for right floodplain corresponding to each elevation HS(K,I).

Note: Cards (30 - 32) are repeated for (NS-1) reaches.

Note: Card (33) is omitted if KFLP = 0, (card (2)).

(33) SNC(K,I) - 8 F 10.0 Format

SNC(I) — Sinuosity coefficient (channel flow-path length/floodplain flow-path length) corresponding to each elevation HS (K,I) for each of the (NS-1) reaches. If left blank, SNC (K,I) is assumed to be 1.0; however, it cannot be omitted if KFLP=1.

*(34) DXM(I) - 8 F 10.0 Format

DXM(I) — Minimum Δx distance (mi) between cross sections used in the computations. If DXM(I) is less than the distance between two adjacent cross sections among the NS cross sections read-in, then intermediate cross sections are created within the program via a linear interpolation procedure. (NS-1) values of DXM(I) are entered on one or more cards (8 values to a card); maximum no. of DXM(I) values is limited to 89; values assigned to DXM(I) should not result in more than 200 cross sections produced by the interpolation procedure. $DXM \leq c\ t_r/(2*MDT)$, default to $DXM = c\ t_r/20$, where c is the approximate speed of the flood wave and t_r is the time of rise (hr) of the routed hydrograph and MDT from card 36. If left blank, program will try to automatically determine the proper value. If a positive first DXM value is entered, a suggested value for a later run will be computed and printed but not used; a negative sign placed before the first value will bypass this computation. If the program aborts before completing the initial condition summary try using the negative sign.

*(35) FKC(I) - 8 F 10.0 Format

FKC(I) — Expansion/contraction coefficient; contraction values vary from 0.05 to 0.4, expansion values vary from -0.05 to -0.75; the larger values are associated with very abrupt changes in cross section along the river. If expansion/contraction effects are negligible, enter 0.0 for FKC(I); (NS-1) values of FKC(I) are entered on one or more cards (8 values to a card); maximum no. of FKC(I) values is limited to 89.

*(36) QMAXD, QLL, DTHM, YDN, SOM, F1I, EPSY, TFI - 8 F 10.0 Format

QMAXD — Estimated maximum discharge (cfs) at downstream extremity of channel/valley reach; can be read in as 0.0 for initial run; subsequent runs can have a value of QMAXD as determined by the routing computations during the initial run; it is required only when QLL is non-zero.

QLL — Maximum lateral outflow (cfs/ft) producing the volume losses experienced by the passage of the dam-break flood wave through the downstream valley; QLL has a negative sign and is computed by Eq. (91) in paper; QLL may be left blank if losses are assumed negligible.

DTHM — Initial Δt time step size (hr); if 0.0 is read in, the value of DTHM is computed by the program; if DTHM<0.0, DTHM represents the divisor MDT for determining the time step (DTH=TFH/MDT) and DTHM is reset to zero. See note on page A-24.

YDN — Parameter for type of downstream boundary. If YDN = 0.0, generated loop rating; if YDN = 0.25, specify single value rating curve of water surface elevation (ft m.s.l.) vs. discharge; if YDN = channel slope (ft/ft), a single value rating curve is assumed and generated using the Manning equation and YDN as the slope; if YDN = 0.5, critical flow such as a waterfall or rapids; if YDN = 0.75, a specified water surface elevation (m.s.l.) such as a tide. When the downstream end of a routing reach is a dam as in options 4, 5, 6, 9, 10, then YDN is read-in as the initial water (pool) elevation just upstream of the dam.

SOM — Slope of downstream channel (ft/mi) for first mile below dam. It is used to compute tailwater via the Manning equation. If left blank, model assumes it is equal to (HS(1,1) - HS(1,NS/3))/[XS(1) - XS(NS/3)) x 5280] in which HS is the invert elevation and XS is the mileage location of sections 1 and NS/3 where NS, XS, HS are defined in cards (20, 24, 25), respectively.

F1I — Theta (Θ) weighting factor in finite difference solution; if left blank, a value of 0.60 is used in program; if 0.5 is used, Θ is set internally to 0.60 and the model is capable of allowing negative flows or any flows less than the initial flow (t=0) to occur by omitting the low-flow filter; if 0.51 is used, Θ is set internally to 0.60 and the routing is done by the diffusion routing method (first two inertial terms in momentum Eq. (9) are omitted) instead of dynamic routing.

EPSY — Convergence criterion for stage (ft) in Newton-Raphson iterative solution of finite difference unsteady flow equations; varies from 0.001 to .1 ft; if left blank, program use 0.01 ft. Also, can be used to specify the exponent m_e used in Eq. (93) in the paper; if EPSY$\leq$0.50, m_e=4; if EPSY>0.5, m_e=EPSY and then EPSY is automatically set to 0.01.

TFI — Time (hr) when time step changes from DTHM to TFH/MDT. See time step note on page A-24.

Note: Cards (37-48) are omitted unless KCG<0, (card (20)).

(37) NPLD - I 10 Format

NPLD — Sequence number of last floodplain compartment on same side of river where first floodplain compartment (FPC) is located. Compartments are consecutively numbered from upstream to downstream on one side of the river and then continuing in sequence at the upstream end on the other side of the river.

(38) NPXI(K), NXPN(K), NQLP(K), PWELV(K), PCWR(K), PEO(K), QMINP(K) - 3 I10, 4 F 10.0 Format

NPXI(K)	Cross section number that coincides with the upstream end of the K^{th} FPC. If (-) value is assigned, the K^{th} FPC elevation is not updated at each iteration within a time step. (This sometimes can enhance the numerical robustness.)
NXPN(K)	Cross section number that coincides with the downstream end of the K^{th} FPC.
NQLP(K)	Parameter indicating if pump discharge within the K^{th} FPC will be specified by a discharge hydrograph; 0 if no, 1 if yes.
PWELV(K)	Average elevation (ft. msl) of crest of weir (levee) along river where inflow to K^{th} FPC occurs.
PCWR(K)	Coefficient of discharge for weir flow from river to K^{th} FPC; ranges in value from 2.6 to 3.2.
PEO(K)	Initial elevation (ft. msl) of water surface in K^{th} FPC at time = 0.
QMINP(K)	Minimum discharge (cfs) of total number of pumps in K^{th} FPC at all times; can be left blank.

(39) PSA(I,K) - 8 F 10.0

PSA(I,K) — Total volume (acre-ft) of K^{th} FPC below each elevation (PEL(I,K)); I index goes from 1 to 8.

(40) PEL(I,K) - 8 F 10.0

PEL(I,K) — Elevation (ft. m.s.l.) associated with each volume (PSA(I,K); elevations start at the lowest and proceed to the highest; I index goes from 1 to 8; last specified elevation should be greater than any expected water elevation within the FPC.

(41) QPU(I,K) - 8 F 10.0

QPU(I,K) Inflow (cfs) to K^{th} FPC other than that transmitted over the weir (levee) from the main river, or levees of adjacent (upstream and/or downstream) FPC; I index goes from 1 to ITEH (card (2)).

Note: Card (42) is omitted if NQLP(K) = 0, (card (38)).

(42) QLP(I,K) - 8 F 10.0

QLP(I,K) Specified total pump discharge (cfs) for K^{th} FPC; I index goes from 1 to ITEH (card (2)).

(43) COFF(I,K) - 8 F 10.0

COFF(I,K) Coefficient of discharge for flow over levee at the downstream end of the K^{th} FPC; coefficient is product of the broad-crested weir coefficient (2.6 to 3.2) and the length (ft) of the weir crest; the coefficient varies with elevation (HCFF(I,K)); I index goes from 1 to 8.

(44) HCFF(I,K) - 8 F 10.0

HCFF(I,K) Elevation (ft. m.s.l.) associated with the discharge coefficients (COFF(I,K)); elevations start at the lowest point along the levee crest and proceed upward; I index goes from 1 to 8.

Note: Cards (38 - 44) are repeated as K index goes from 1 to ABS(KCG). Most downstream FPC joins the river at cross-section number NXPN (NPLD) and/or NXPN (ABSCKCG).

(45) NPM - I 10

NPM Total number of pumps in all of the FPC's; maximum of 25 allowed.

Note: Cards (46 - 48) are omitted if NPM=0 (card (45)).

(46) IPMPL(L), NXPO(L), PEMN(L), PEMX(L) - 2 I 10, 2 F 10.0

IPMPL(L)	Number of the K^{th} FPC in which the L^{th} pump is located; L goes from 1 to NPM.
NXPO(L)	Number of the cross section immediately upstream of Δx reach where the L^{th} pump discharges into main river.
PEMN(L)	Elevation (ft. m.s.l.) of water in K^{th} FPC when L^{th} pump starts pumping.
PEMX(L)	Elevation (ft. m.s.l.) of water in K^{th} FPC when L^{th} pump stops pumping.

Note: Cards (47 - 48) are omitted if NQLP(K) = 1, (card (38)).

(47) DHP(I,L) - 8 F 10.0

DHP(I,L)	Head (ft) associated with L^{th} pump rating curve; I index goes from 1 to 8; head starts at smallest and proceeds to greatest; negative head may be specified.

(48) OP(I,L) - 8 F 10.0

OP(I,L)	Pump discharge (cfs) associated with L^{th} pump rating curve; I index goes from 1 to 8; each value is associated with its corresponding DHP(I,L) value.

Note: Repeat cards (46 - 48) as L index goes from 1 to NPM.

Note: Omit cards (49 - 50) if LQ=0 (card (20)).

(49) LQX(K) - 8 I 10 Format

LQX(K)	Number of cross section immediately upstream of lateral inflow/outflow; K goes from 1 to LQ (card (20)). If LQX(K) is specified as a negative number, this indicates that the reach may have outflow via broad-crested weir flow.

(50) QL(L,K) - 8 F 10.0 Format

QL(L,K)	Lateral inflow (cfs) for K^{th} lateral inflow point; L index goes from 1 to ITEH (card (2)); ordinates of lateral inflow hydrograph have same times as those of reservoir inflow hydrograph (QI(L)) on card (18)); K index goes from 1 to LQ.

If LQX(K) is negative, three values only are specified on card (50) according to a 3 F 10.2 format. The first, (WELV(K)), is the crest elevation (msl) at which overflow occurs (this represents the average crest elevation along the reach). The second, (CWR(K)), is the discharge coefficient ranging in value from 2.6 to 3.2 with 3.0 a most common value. The third, (HLRS(K)), is the difference in the max and min crest elevations along the reach (this is sometimes useful to prevent numerical problems with sudden large outflows when the levee is first overtopped); it may be left blank in which case the crest is assumed to be level.

Note: Cards (51 - 52) are omitted unless YDN = 0.25, (card (36)).

(51) RH(K) - 8 F 10.0 Format

RH(K) Elevation (ft. m.s.l.) points on single-value rating curve for downstream boundary, K index goes from 1 to maximum of 8.

(52) RQ(K) - 8 F 10.0 Format

RQ(K) Discharge (cfs) associated with elevation points on single value rating curve for downstream boundary; K goes from 1 to maximum of 8.

Note: Cards (53 - 54) are omitted unless YDN = 0.75, (card (36)).

(53) STN(K) - 8 F 10.0 Format

STN(K) Specified water surface elevation (ft. m.s.l.) at downstream boundary such as a tide; K goes from 1 to ITEH.

(54) TTN(K) - 8 F 10.0 Format

TTN(K) Time (hrs) associated with STN(K); K goes from 1 to ITEH.

Note: Cards (55 - 56) are omitted unless KSL $\geq$ 1, (card (15)).

(55) LSI, LSN, LSL, LSM, LSU, TSL, ALPHA, POR - 5 I10, 3 F10.3 Format

LSI	Sequential number of most upstream cross section where landlisde begins; must be > 1.
LSN	Sequential number of most downstream section where landslide ends; must be < NS.
LSL	Sequential number of elevation in topwidth-elevation table where landslide first begins; this must be $\geq$ 3.
LSM	Sequential number of elevation in topwidth-elevation table where landslide is thickest; must be > LSL.
LSU	Sequential number of elevation in topwidth-elevation table where landslide ends; this must be > LSM and $\leq$ NCS.
TSL	Time (hr) of duration of landslide (usually in the range of 15 seconds to a few minutes).
ALPHA	Angle (degrees) of repose that deposited material from the landslide assumes in the bottom of reservoir at its center or mid-point of the width of the reservoir.
POR	Porosity of landslide material; decimal fraction.

(56) THKSL(k) - 8 F10.1 Format

THKSL(k)	Thickness (ft., measured into bank of reservoir) of landslide mass at the LSMth elevation in the topwidth table; K index goes from 1 to (LSN - LSI + 1).

<u>Note</u>: Cards (57 - 60) are omitted unless KCG>0, (card (20)).

(57) ICG(K) - 8 I 10 Format

ICG(K)	Parameter indicating if a dam has time-dependent movable gate flow; if yes, ICG(K)=1; if no, ICG(K)=0; K goes from 1 to M, where M=MULDAM if MULDAM $\geq$ 1 and M=1 if MULDAM=0.

(58) CGCG(L,K) - 8 F 10.0 Format

CGCG(L,K)	Spillway gate width (ft) opened at time TCG(L,K); L goes from 1 to KCG (see card 20); and K goes from 1 to the total number of dams having time-dependent gate control; CG on card (9) must be equal or greater than 1.0.

(59) GBL(L,K) - 8 F 10.0 Format

GBL(L,K) Distance (ft) from bottom of gate to gate sill (HGT on card(9)); this distance is time dependent and is associated with the time array TCG(L,K); L and K indices are same as described on card (58).

(60) TCG(L,K) - 8 F 10.0 Format

TCG(L,K) Time (hrs) associated with CGCG(L,K); L goes from 1 to KCG; and K goes from 1 to the total number of dams having time-dependent movable gate control.

Note: Cards (61-64) are omitted unless option 9 or 10 (see page A-22) is used.

(61) Z, YBMIN, BB, TFH - 4 F 10.0 Format

Z	Side slope (1:vertical to Z:horizontal) of breach of downstream dam.
YBMIN	Lowest elevation (ft) that bottom of breach reaches; elevation is referenced to m.s.l. datum.
BB	Width (ft) of base of breach of downstream dam.
TFH	Time (hr) from beginning of breach formation of downstream dam until it reaches its maximum size.

(62) HF, HD, HSP, HGT, CS, CG, CDO, QT - 8 F 10.0 Format

HF	Elevation (ft) of water when failure of downstream dam commences; elevation is referenced to m.s.l. datum.
HD	Elevation (ft) of top of downstream dam; elevation is referenced to m.s.l. datum.
HSP	Elevation (ft) of uncontrolled spillway crest; elevation is referenced to m.s.l. datum.
HGT	Elevation (ft) of center of gate openings; elevation is referenced to m.s.l. datum.
CS	Discharge coefficient for uncontrolled spillway; it is equal to the coefficient of discharge (2.6-3.2) times the length (ft) of the spillway.
CG	Discharge coefficient for gate flow; it is equal to the coefficient of discharge (0.60-0.80) times the area of gates.
CDO	Discharge coefficient for uncontrolled weir flow over the top of the downstream dam; it is equal to the coefficient of discharge (2.6-3.2) times the length of the downstream dam crest (ft) less the length of the uncontrolled spillway and gates.
QT	Discharges (cfs) through turbines; this flow is defined the same as on card (9).

<u>Note</u>: Cards (63 - 64) are read-in only if either HSP is non-zero and CS is zero, or HGT is non-zero and CG is zero. This option allows a rating curve to be used for either the uncontrolled spillway or submerged gate rather than an equation for each using a constant discharge coefficient as in Eq. (32).

(63) QSPILL(K,1) - 8 F 10.0 Format

QSPILL(K,1) Flow (cfs) of spillway or gate rating curve; K goes from 1 to maximum of 8.

(64) HEAD(K,1) - 8 F 10.0 Format

HEAD(K,1) Head (ft) above spillway crest or gate center; head is associated with spillway flow or gate flow in rating curve.

(65) UPSH, SOM, CMN - 3 F 10.0 Format

UPSH Dummy variable, leave blank.

SOM Slope of downstream channel (ft/mi) for first few miles below dam. (Controls tailwater elevation computed via Manning equation.)

CMN Average Manning's n for downstream channel for first few miles below dam. (Controls tailwater elevation computed via Manning equation.)

Note: The program has the capability of simulating a total of 12 different cases. These are outlined as follows:

Program Options

Option 1: Reservoir storage routing to compute outflow hydrograph from reservoir with subcritical dynamic routing of outflow hydrograph through entire length of downstream valley; tailwater depth (used in correcting dam outflow for tailwater submergence effects) computed by Manning equation applied to tailwater section--KKN=1, KUI=0, MULDAM=0, KSUPC=0. Input data cards--1-4, 6-60.

Option 2: Reservoir storage routing to compute outflow hydrograph from reservoir with supercritical dynamic routing of outflow hydrograph through entire length of downstream valley--KKN=1, KUI=0, MULDAM=0, KSUPC=1.
Input data cards--1-4, 6-60.

Option 3: Reservoir storage routing to compute outflow hydrograph from reservoir with supercritical dynamic routing of outflow hydrograph through upstream portion of downstream valley and subcritical dynamic routing through downstream portion of downstream valley--KKN=2, KUI=0, MULDAM=0, KSUPC=1.
Input data cards--1-4, 6-60, 20-54.

Option 4: Same as Option 1 except reservoir dynamic routing to compute outflow hydrograph from reservoir--KKN=2, KUI=1, MULDAM=0, KSUPC=0.
Input data cards--1-4, 8-60, 65, 20-54.

Option 5: Same as Option 2 except reservoir dynamic routing to compute outflow hydrograph from reservoir--KKN=2, KUI=1, MULDAM=0, KSUPC=1.
Input data cards--1-4, 8-60, 65, 20-54

Option 6: Same as Option 3 except reservoir dynamic routing to compute outflow hydrograph from reservoir--KKN=3, KUI=1, MULDAM=0, KSUPC=1.
Input data cards--1-4, 8-60, 65, 20-54, 20-54.

Option 7: Subcritical dynamic routing of input hydrograph through a channel/valley--KKN=9, KUI=0, MULDAM=0, KSUPC=0,
Input data cards--1-4, 15-54.

Option 8:	Supercritical dynamic routing of input hydrograph through a channel/valley--KKN=9, KUI=0, MULDAM=0, KSUPC=1. Input data cards--1-4, 15-54.
Option 9: "Sequential Method"	Reservoir storage routing to compute outflow hydrograph from reservoir with subcritical dynamic routing of outflow hydrograph through downstream channel/reservoir having a dam which may fail; tailwater depths for submergence correction of outflow from dam computed using the Manning equation--KKN=number of dams, KUI=0, MULDAM=1, KSUPC=0, 2, 3, 4. Input data cards--1-4, 6-65, 20-65, ... 20-54.
Option 10: "Sequential Method"	Reservoir dynamic routing to compute outflow hydrograph from reservoir with subcritical dynamic routing of outflow hydrograph through downstream channel/reservoir having a dam which may fail; tailwater depths used for submergence correction of outflow from dam computed using the Manning equation--KKN=1+number of dams, KUI=1, MULDAM=1, KSUPC=0, 2, 3, 4. Input data cards--1-4, 8-65, 20-65, ... 20-54.
Option 11: "Simultaneous Method"	Simultaneous computation method for single dam or bridge (structure) using dynamic routing in the reach upstream of the structure and downstream of the structure with special internal boundary conditions for flow thru the structure--KKN=1, KUI=1, MULDAM=1, KSUPC=0. Input data cards--1-5, 8-14, 15-60. See note on page A-23 for input variables for bridge/embankment.
Option 12: "Simultaneous Method"	Simultaneous computation method for multiple dams and/or bridges (structures) using dynamic routing for all reaches with special internal boundary conditions for flow thru each structure--KKN=1, KUI=1, MULDAM=no. of dams and/or bridges, KSUPC=0. Input data cards--1-5, 8-9, 8-9, 8-9, ... 10-14, 10-14, 10-14, ... 15-60. See note on page A-23 for input variables for bridge/embankments.

"SIMULTANEOUS METHOD" OF COMPUTATION OF COMPLETE SYSTEM:

This option treats the upstream reservoir, any intermediate reservoir and dam or bridge, and the downstream channel as one system. Cross sections are numbered consecutively from the very upstream end of the most upstream reservoir to the downstream extremity of the downstream channel. Cross sections are specified just upstream and downstream of each dam. This option is most useful for problems in which the tailwater below a dam is affected by backwater from downstream dams or other constrictions. It is necessary when using this option to specify KKN=1, KUI=1, MULDAM $\geq$ 1, and to read-in card (5). Also, cards (6) and (7) are omitted, and cards (8) and (9) are repeated for each dam in the system. Cards (61-65) are not applicable for this option. MULDAM is defined as the number of dams and/or bridges in the system.

BRIDGE COMPUTATION

The simultaneous method can be used for either multiple dams and/or bridges. Cards (8) and (9) are used to describe the flow thru and across the bridge/embankment. The embankment may be allowed to breach. If breaching is not considered possible, HF on card (9) is set to a very large value so that the water surface will not reach it. On card (8), RLM and YO must be left blank; other variables on card (8) are associated with the breach of the embankment and are defined essentially the same as shown on page A-3. The variables other than HF on card (9) are defined as follows: HD--height (ft m.s.l.) of crest of uppermost portion of road embankment; HSPD--length (ft) of crest of uppermost portion of road embankment measured across valley and perpendicular to flow; HGTD--height (ft m.s.l.) of crest of lower portion (emergency overflow) of road embankment (if non-existent, leave blank); CSD--length (ft) of crest of lower portion of road embankment measured across valley and perpendicular to flow; CGD--width of top of road embankment as measured parallel to flow; CDOD--coefficient of discharge of flow thru bridge opening (see: Chow, "Open-Channel Hydraulics" pp. 476-490); QT--time step to be used when the upper road embankment is overtopped. QT is only needed when a bridge is in the routing reach and when the inflow hydrograph is a slowly rising hydrograph that overtops the embankment. It should be left blank at all other times. If it is left blank when needed, the default value is 0.5 hr. Instead of reading in the length of the upper road embankment, a table of length of embankment (ft) (QSPILL) vs. head (HEAD) above lowest elevation of embankment (HD) may be read in on cards (13) and (14) respectively. HSPD is then read in as zero.

ROUTING SPECIFIED INFLOW HYDROGRAPH (Options 7 and 8)

Options (7) and (8) are for routing a specified inflow hydrograph through the downstream valley, i.e., there is no upstream reservoir and associated outflow hydrograph as computed by DAMBRK. These options do not enable the treatment of bridges or dams located along the downstream valley; however, option (11) or (12) can be used for this purpose.

LEVEL-POOL ROUTING USING OPTIONS 11 AND 12

The storage routing (level-pool) technique may also be used simultaneously with the dynamic routing technique for simulating the unsteady flow through the downstream channel/valley. Eq. (68) is used as the upstream boundary condition and the dam is treated as an internal boundary via Eqs. (30-31). The advantages of this combination of the two routing techniques within a simultaneous computation method are: (1) simple routing technique for reservoir, (2) dynamic routing for downstream dam-break hydrograph, and (3) more accurate computation of tailwater elevation than via the Manning equation. Use KKN=KUI=MULDAM=1 and IDAM (1) = 1, i.e., the first cross section is located just upstream of the first dam. Level-pool routing can only be used for the first or upstream reservoir. If IDAM (1) = -1, everything is the same as this option except dynamic routing is used in the most upstream reservoir. Reservoir cross sections are automatically created from the surface area-elevation table (cards (6-7) and RLM (card (8)).

FLOODPLAIN COMPARTMENTS (FPC)

Each FPC may have several consecutive Δx-reaches in which flow from the river enters the FPC or, if the differential head favors the FPC, the flow goes from the FPC to the river. These Δx-reaches are designated by NXPI(K) to NXPN(K), where the K index goes from 1 to the total number of FPC's (KCG).

Each FPC may have only one Δx-reach in which the FPC pump(s) return water to the river. The Δx-reach is designated by NXPO(L), where the L index goes from 1 to the total number of pumps (NPM).

A particular Δx-reach can be connected to two FPC's, one on each side of the river.

A FPC may pass flow to an adjacent FPC via broad-crested weir flow with submergence correction. Also, the most downstream FPC may pass flow on downstream via overtopping weir flow. FPC's are numbered from upstream to downstream, commencing on one side of the river and then continuing on the other side of the river.

TIME STEP SELECTION

The time step size used to route the hydrograph through the downstream channel-valley can be user controlled with the DTHM and TFI parameters on card (36). If a constant time step is desired, the user reads in DTHM (time step size) and leaves TFI blank.

If DTHM and TFI are both read in as zero, the model will generate an initial time step size based on the inflow hydrograph - TP/MDT where TP is the time from start of rise to peak of the hydrograph and MDT is assumed to be 20 unless specified differently by reading in a negative DTHM value, in which case MDT = |DTHM|. This time step is used until time TFI (the time just prior to dam failure) is exceeded. If KUI=0 (card 2), this value is computed as the time to peak of the outflow hydrograph minus the time to failure. If KUI=1, TFI is set equal to TEH (card 15). If the time exceeds TFI or if the dam fails, the time step is cut back to TFH/MDT. If DTHM and TFI are read in as nonzero values, the DTHM is used until time TFI is exceeded and then the time step is cut back to TFH/MDT.

If TFI is read in as a nonzero value and DTHM is read in as zero, the model will compute DTHM=TP/MDT and use that time step until TFI is exceeded and then cut back to TFH/MDT.

ALPHABETICAL LISTING OF DATA INPUT

VARIABLE	CARD NO.	DEFINITION OF VARIABLE
ALPHA	- 55	Angle of repose for landslide option
BB	- 8	Width of the base of the breach
BREX	- 15	Exponent used in the development of breach
BS(K,I)	- 26	Topwidth of active flow portion of channel valley section
BSBR(L,K)	- 11	Width associated with bridge/culvert opening
BSL(K,I)	- 27	Topwidth of active flow portion of left floodplain
BSR(K,I)	- 28	Topwidth of active flow portion of right floodplain
BSS(K,I)	- 29	Topwidth of off-channel storage portion of channel-valley section
CDO	- 9	Discharge coefficient for uncontrolled weir flow over the top of the dam
CG	- 9	Discharge coefficient for gate flow
CGCG(L,K)	- 58	Spillway gate width opened at time TCG(L,K)
CM(K,I)	- 30	Manning n for channel
CML(K,I)	- 31	Manning n for left floodplain
CMN	- 65	Average Manning's n for downstream channel for first few miles below dam
CMR(K,I)	- 32	Manning n for right floodplain
COFF(I,K)	- 43	Coefficient of discharge for flow over levee separating the Kth & Kth+1 floodplain compartment (FPC)
CS	- 9	Discharge coefficient for uncontrolled spillway
CWR(K)	- 50	Discharge coefficient for bulk lateral outflow
DATUM	- 8	Elevation of bottom of dam
DHF	- 15	Interval between QI(K) input hydrograph ordinates
DHP(I,L)	- 47	Head associated with OP(I,L), pump discharge
DTHM	- 36	Initial time step size
DXM(I)	- 34	Minimum distance interval between sections used in computations
EPSY	- 36	Convergence criterion for stage in Newton Iteration technique
FKC(I)	- 35	Contraction/expansion coefficient
FSTG(I)	- 24	Elevation at which flooding commences
F1I	- 36	Theta weighting factor in finite-difference solution
GAMA	- 17	Ratio of time from initial steady flow to center of gravity of inflow hydrograph to time to peak of inflow hydrograph
GBL(L,K)	- 59	Distance from bottom of gate to gate sill for time-dependent gates
HCFF(I,K)	- 44	Elevation associated with the discharge coefficients (COFF(I,K)
HD	- 9	Elevation of top of dam
HEAD(KL,K)	- 14	Head above spillway crest of gate center (associated with QSPILL (KL,K)
HF	- 9	Elevation of water when failure commences
HGT	- 9	Elevation of center of gate openings

Appendix B-2

HLRS(K)	- 50	Difference in maximum & minimum levee crest elevations along Δx reach for bulk lateral outflow option
HS(K,I)	- 25	Elevation corresponding to each topwidth (BS(K))
HSA(K)	- 7	Elevation of reservoir corresponding to each SA(K) value
HSBR(L,K)	- 10	Elevation associated with widths of bridge/culvert opening
HSP	- 9	Elevation of uncontrolled spillway crest
ICG(K)	- 57	Parameter indicating if dam has time-dependent gate flow
IDAM(K)	- 5	Sequence no. of section coincident with the upstream face of each dam
IOPUT(K)	- 4	Print parameter to suppress various output data
IPMPL(L)	- 46	No. of Kth FPC in which the Lth pump is located
ITEH	- 2	No. of inflow hydrograph ordinates
IWF	- 15	Parameter denoting dry-bed routing
JNK	- 20	Output print parameter
KCG	- 20	No. of ordinates in time-dependent spillway gate control curve
KDMP	- 2	Printing parameter
KFLP	- 2	Floodplain option parameter
KKN	- 2	Parameter to control options 1-12
KPRES	- 15	Parameter indicating method of computing hydraulic radius
KSA	- 20	Parameter indicating type of cross-section smoothing
KSL	- 2	Landslide option parameter
KSUPC	- 20	Parameter indicating type of flow
KUI	- 2	Parameter to control options 1-12
LQ	- 20	No. of lateral flows
LQX(I)	- 49	No. of section immediately upstream of lateral flow
LSI	- 55	Sequence no. of most upstream section where landslide begins
LSL	- 55	Sequence no. of elevation in BS vs HS table where landslide begins
LSM	- 55	Sequence no. of elevation in BS vs HS table where landslide is thickest
LSN	- 55	Sequence no. of most downstream section where landslide ends
LSU	- 55	Sequence no. of elevation in BS vs HS table where landslide ends
MDAM	- 1	Name of dam
MESAGE	- 1	Agency address
METRIC	- 15	Parameter indicating in input/output is in Englist or metric units
MNAME	- 1	Agency name
MUD	- 15	Parameter denoting mud/debris flow
MULDAM	- 2	Parameter to control options 1-12
MRVR	- 1	Name of reservoir
NCS	- 20	Maximum no. of top widths used to describe each section
NDSM(K)	- 23	Downstream cross section no. of Kth smoothing reach
NPLD	- 37	No. of last floodplain compartment (FPC) on same side of river where 1st floodplain compartment is located
NPM	- 45	Total no. of pumps in all of the FPC's
NPRT	- 2	No. of sections for which hydraulic info will be printed
NPT(K)	- 3	Sequence no. of section at which hydraulic info will be printed

NQLP(K)	- 38	Parameter indicating if pump discharge within the Kth FPC will be specified by a discharge hydrograph
NS	- 20	No. of cross sections in the routing reach
NSMR	- 22	No. of separate smoothing reaches within the total routing reach
NT(K)	- 21	Sequence of section at which hydrographs will be plotted
NTSM	- 22	Parameter indicating type of smoothing
NTT	- 20	Total no. of plotting stations
NUSM(K)	- 23	Upstream cross section no. of Kth smoothing reach
NXPI(K)	- 38	No. of section immediately upstream of 1st reach where inflow to Kth FPC occurs
NXPN(K)	- 38	No. of section immediately upstream of last reach where inflow to Kth FPC occurs
NXPO(L)	- 46	No. of section immediately upstream of reach where Lth pump discharges into main river
OP(I,L)	- 48	Pump discharge associated with Lth pump rating curve
PCWR(K)	- 38	Coefficient of discharge for weir flow in FPC option
PEL(I,K)	- 40	Elevation associated with each volume PSA(I,K)
PEMN(L)	- 46	Elevation of water in Kth FPC when Lth pump starts pumping
PEMX(L)	- 46	Elevation of water in Kth FPC when Lth pump stops pumping
PEO(K)	- 38	Initial elevation of water surface in Kth FPC at time =0
POR	- 55	Porosity of landslide material
POWR	- 16	Exponent in power function representing the stress-rate of strain relation
PSA(I,K)	- 39	Total volume of Kth FPC below each elevation
PWELV(K)	- 38	Average elevation of crest of weir along reach where inflow to Kth FPC occurs
QI(K)	- 18	Inflow hydrograph at the upstream end of the routing reach
QL(L,K)	- 50	Lateral inflow
QLL	- 36	Maximum lateral outflow producing volume losses
QLP(I,K)	- 42	Specified total pump discharge for Kth FPC
QMAXD	- 36	Estimate maximum discharge at downstream end of chanel-valley reach
QMINP(K)	- 38	Maximum discharge of total no. of pumps in Kth FPC at all times
QO	- 17	Initial steady discharge
QPU(I,K)	- 41	Inflow to Kth FPC other than that transmitted as weir flow
QSPILL(KL,K)	- 13	Flow of spillway or gate rating curve
QT	- 9	Discharge through the turbines
RH(K)	- 51	Elevation points on single-value rating curve
RHO	- 17	Ratio of peak flow to initial flow of inflow hydrograph
RLM	- 8	Length of reservoir
RQ(K)	- 52	Discharge associated with RH(K)
SA(K)	- 6	Surface area or volume of reservoir at elevation HSA(K)
SHR	- 16	Initial yield stress or shear strength of mud/debris fluid
SMF	- 22	Smoothing factor
SNC(K,I)	- 33	Sinuosity coefficient
SOM	- 36	Slope of downstream channel for 1st mile below dam
STN(K)	- 53	Specified water surface elevation hydrograph at downstream boundary
TCG(L,K)	- 60	Time associated with CGCG(L,K), width of movable gate
TEH	- 15	Time from beginning of routing until routing is terminated

TFH	- 8	Time of failure of the structure
TFI	- 36	Time when time step changes form DTHM to TFH/MDT
THKSL(K)	- 56	Thickness of landslide mass
TI(K)	- 19	Time associated with each inflow hydrograph point
TPG	- 17	Time from inital flow to peak flow of inflow
TSL	- 55	Time of duration of landslide
TTN(K)	- 54	Time associated with STN(K)
UPSH	- 65	Dummy variable
UW	- 16	Unit weight of mud/debris fluid
VIS	- 16	Dynamic viscosity of mud/debris fluid
VOL	- 8	Parameter indicating if SA(K) is surface area or volume
WELV(K)	- 50	Crest elevation at which overflow occurs for bulk lateral outflow option
XS(I)	- 24	Location of sections used to describe downstream channel valley
YBMIN	- 8	Lowest elevation that the bottom breach reaches
YDN	- 36	Parameter for type of downstream boundary
YO	- 8	Initial water surface elevation in the reservoir
Z	- 8	Side slope of breach

APPENDIX C -- New Features of DAMBRK: Version 6/20/88

1. Subcritical/Supercritical mixed flow solution algorithm and supercritical flow solution algorithm

2. Conveyance option for simulating wide, flat floodplains

3. Sinuosity coefficient for meander effect

4. Mixed free-surface/pressurized flow capability

5. Momentum coefficient (β) for non-uniform velocity across section

6. Internal viscous dissipation slope (S_i) for non-Newtonian mud/debris or mine-tailings flow

7. Hydraulic radius option to use cross-sectional topwidth or perimeter

8. Automatic determination of Δx computational distance step according to: (a) $\Delta x \leq C \Delta t$, (b) limitations imposed by rapidly expanding/contracting reaches, and (c) sudden large changes in bottom slope.

9. Bridge computation that uses friction losses in available energy head and separate topwidth-elevation table for bridge opening

10. Breach development may be linear on nonlinear

11. Breach may be confined to the spillway section and spillway flow is reduced as breach flow increases

12. Breach may commence formation when either a specified elevation is attained by the reservoir surface elevation or a specified time after beginning of simulation is attained

13. Breach via a piping failure may occur after beginning of simulation according to when either specified creterion is attained

14. Dam crest length may be a function of elevation for dams with uneven crests

15. Option to create cross sections for reservoir (to allow dynamic routing) when only a surface-area elevation relation is specified

16. Modified option to convert reservoir volume to surface area so as to start at top of reservoir and proceed to the bottom

17. Option to allow input/output in English or metric units

18. Modified automatic procedure for coping with numerical nonconvergence by reducing number of solution attempts

19. Modified low flow filter to use critical depth associated with initial flows in reservoirs as the minimum possible computed depth

20. Added more print/output options

21. PC micro-computer load module run time reduced by approximately 40% due to a more efficient new compiler which is counter-acted by increased run time due to overlay requirements to fit new model on 640K micro.

22. Modified landslide option to allow interpolation of additional cross sections within landslide reach

23. Modified time-dependent movable gate option by: (a) using specified gate widths, and (b) computing the discharge coefficient

24. Computational scheme option to allow dry-bed routing of mud flows in moderate to flat sloping channels

25. Modified downstream boundary condition of channel-control loop-rating to include inertial terms in the energy slope approximation used in the Manning equation

26. May create additional cross sections in reaches where computed lateral outflows occur, e.g., reaches connecting with floodplain compartments, reaches in which overtopping levee flow occurs

27. Modified initial condition computations to produce spatially varied steady flow in reservoirs where inflow and outflow differ

EXAMPLE-1 OPT: 11

1	1	1	5	3	0		
1000001100							
2							
0.	5288.5	1.04	5027.	81.	1.43	5027.	0.
5289.0	5288.5	0.	0.	0.	0.	300.	13000.
0.	55.						
13000.	50000.	13000.					
0.	1.	55.					
14	5	6	4	0	0		
1	2	3	5	12	14		
0.	5230.						
5220.	5230.	5240.	5250.	5290.			
200.	200.	200.	200.	200.			
0.							
16.0	5037.						
5027.	5037.	5100.	5200.	5290.			
200.	500.	1000.	1250.	1350.			
0.							
16.01	5047.						
5027.	5037.	5051.	5107.	5125.			
0.	590.	820.	1130.	1200.			
21.01	4985.						
4965.	4980.	5015.	5020.	5030.			
0.	850.	1100.	1200.	1300.			
0.	0.	3500.	4300.	5300.			
24.51	4946.						
4920.	4930.	4942.	4953.	4958.			
0.	800.	4000.	11000.	15000.			
0.	0.	0.	7000.	10000.			
32.01	4830.						
4817.	4827.	4845.	4847.	4852.			
0.	884.	4000.	11000.	22000.			
0.	0.	30000.	27000.	25000.			
38.51	4820.						
4805.	4812.	4814.	4825.	4830.			
0.	1000.	1200.	11000.	16000.			
0.	0.	0.	6000.	8000.			
43.51	4800.						
4788.	4792.	4802.	4808.	4810.			
0.	286.	7000.	10000.	11000.			
0.	0.	0.	3500.	5000.			
48.51	4777.						
4762.	4774.	4777.	4780.	4785.			
0.	352.	5000.	10000.	18000.			
0.	0.	9000.	16000.	24000.			
53.51	4767.						
4752.	4763.	4768.	4773.	4778.			
0.	450.	3500.	6000.	9000.			

0.	0.	4000.	8500.	12000.			
57.01	4756.						
4736.	4756.	4761.	4763.	4768.			
0.	540.	2000.	4000.	6000.			
0.	0.	3700.	3700.	5500.			
59.01	4749.						
4729.	4737.	4749.	4757.	4759.			
0.	250.	587.	1750.	2000.			
0.	0.	0.	1500.	2000.			
67.51	4674.						
4654.	4659.	4668.	4678.	4683.			
0.	70.	352.	400.	420.			
0.							
75.51	4612.						
4601.	4604.	4606.	4615.	4620.			
0.	245.	450.	500.	520.			
0.							
.06	.06	.05	.04	.04			
.03	.03	.03	.03	.03			
.08	.08	.08	.08	.08			
.05	.05	.05	.05	.05			
.031	.031	.031	.031	.031			
.034	.034	.034	.034	.034			
.038	.038	.038	.038	.038			
.037	.037	.037	.037	.037			
.034	.034	.034	.034	.034			
.034	.034	.034	.034	.034			
.034	.034	.034	.034	.034			
.036	.036	.036	.036	.036			
.036	.036	.036	.036	.036			
2.	2.	.5	.5	.5	.75	1.	1.
1.	1.1	1.0	1.0	1.4			
			-0.9			0.1	-0.5
0.							
0.	0.	0.	0.				

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-1
ON
OPT: 11

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

****
****
***   ***
*** SUMMARY OF INPUT DATA ***
***   ***
****
****

INPUT CONTROL PARAMETERS FOR EXAMPLE-1

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	1
MULTIPLE DAM INDICATOR	MULDAM	1
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

IOPUT= 1 0 0 0 0 0 1 1 0 0 0 0

IDAM= 2

DAM NUMBER 1

EXAMPLE-1 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
***	*****	********	********
ELEVATION OF WATER SURFACE	FEET	YO	5288.50
SIDE SLOPE OF BREACH		Z	1.04
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	5027.00
WIDTH OF BASE OF BREACH	FEET	BB	81.00
TIME TO MAXIMUM BREACH SIZE	HR	TFH	1.43
ELEVATION OF WATER WHEN BREACHED	FEET	HF	5289.00
ELEVATION OF TOP OF DAM	FEET	HD	5288.50
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	300.00
DISCHARGE THRU TURBINES	CFS	QT	13000.00

DHF (INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH (TIME AT WHICH COMPUTATIONS TERMINATE) = 55.0000 HRS.
BREX (BREACH EXPONENT) = 0.000
MUD (MUD FLOW OPTION) = 0
IWF (TYPE OF WAVE FRONT TRACKING) = 0
KPRES (WETTED PERIMETER OPTION) = 0
KSL (LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-1
****

13000.00 50000.00 13000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 55.0000

CROSS-SECTIONAL PARAMETERS FOR OPT: 11
BELOW EXAMPLE-1

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF CROSS-SECTIONS	NS	14
MAXIMUM NUMBER OF TOP WIDTHS	NCS	5
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 3 5 12 14

CROSS-SECTIONAL VARIABLES FOR OPT: 11
BELOW EXAMPLE-1

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 5230.00

HS ...	5220.0	5230.0	5240.0	5250.0	5290.0
BS ...	200.0	200.0	200.0	200.0	200.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 16.000 FSTG(I) = 5037.00

HS ...	5027.0	5037.0	5100.0	5200.0	5290.0
BS ...	200.0	500.0	1000.0	1250.0	1350.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 3
****

XS(I) = 16.010 FSTG(I) = 5047.00

HS ...	5027.0	5037.0	5051.0	5107.0	5125.0
BS ...	0.0	590.0	820.0	1130.0	1200.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 4
****

XS(I) = 21.010 FSTG(I) = 4985.00

HS ...	4965.0	4980.0	5015.0	5020.0	5030.0
BS ...	0.0	850.0	1100.0	1200.0	1300.0
BSS ...	0.0	0.0	3500.0	4300.0	5300.0

1

CROSS-SECTION NUMBER 5
****

XS(I) = 24.510 FSTG(I) = 4946.00

HS ...	4920.0	4930.0	4942.0	4953.0	4958.0
BS ...	0.0	800.0	4000.0	11000.0	15000.0
BSS ...	0.0	0.0	0.0	7000.0	10000.0

CROSS-SECTION NUMBER 6
****

XS(I) = 32.010 FSTG(I) = 4830.00

HS ...	4817.0	4827.0	4845.0	4847.0	4852.0
BS ...	0.0	884.0	4000.0	11000.0	22000.0
BSS ...	0.0	0.0	30000.0	27000.0	25000.0

CROSS-SECTION NUMBER 7
****

XS(I) = 38.510 FSTG(I) = 4820.00

HS ...	4805.0	4812.0	4814.0	4825.0	4830.0
BS ...	0.0	1000.0	1200.0	11000.0	16000.0
BSS ...	0.0	0.0	0.0	6000.0	8000.0

CROSS-SECTION NUMBER 8
****

XS(I) = 43.510 FSTG(I) = 4800.00

HS ...	4788.0	4792.0	4802.0	4808.0	4810.0
BS ...	0.0	286.0	7000.0	10000.0	11000.0
BSS ...	0.0	0.0	0.0	3500.0	5000.0

1

CROSS-SECTION NUMBER 9
****

XS(I) = 48.510 FSTG(I) = 4777.00

HS ...	4762.0	4774.0	4777.0	4780.0	4785.0
BS ...	0.0	352.0	5000.0	10000.0	18000.0
BSS ...	0.0	0.0	9000.0	16000.0	24000.0

CROSS-SECTION NUMBER 10
****

XS(I) = 53.510 FSTG(I) = 4767.00

HS ...	4752.0	4763.0	4768.0	4773.0	4778.0
BS ...	0.0	450.0	3500.0	6000.0	9000.0
BSS ...	0.0	0.0	4000.0	8500.0	12000.0

CROSS-SECTION NUMBER 11
****

XS(I) = 57.010 FSTG(I) = 4756.00

HS ...	4736.0	4756.0	4761.0	4763.0	4768.0
BS ...	0.0	540.0	2000.0	4000.0	6000.0
BSS ...	0.0	0.0	3700.0	3700.0	5500.0

CROSS-SECTION NUMBER 12
****

XS(I) = 59.010 FSTG(I) = 4749.00

HS ...	4729.0	4737.0	4749.0	4757.0	4759.0
BS ...	0.0	250.0	587.0	1750.0	2000.0
BSS ...	0.0	0.0	0.0	1500.0	2000.0

1

CROSS-SECTION NUMBER 13
****

XS(I) = 67.510 FSTG(I) = 4674.00

HS ...	4654.0	4659.0	4668.0	4678.0	4683.0
BS ...	0.0	70.0	352.0	400.0	420.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 14
****

XS(I) = 75.510 FSTG(I) = 4612.00

HS ...	4601.0	4604.0	4606.0	4615.0	4620.0
BS ...	0.0	245.0	450.0	500.0	520.0
BSS ...	0.0	0.0	0.0	0.0	0.0

HS(1, 3) IS GREATER THAN HS(1, 2).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE BASE FLOW IS QUITE SMALL.

1

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1	...	0.060	0.060	0.050	0.040	0.040
REACH 2	...	0.030	0.030	0.030	0.030	0.030
REACH 3	...	0.080	0.080	0.080	0.080	0.080
REACH 4	...	0.050	0.050	0.050	0.050	0.050
REACH 5	...	0.031	0.031	0.031	0.031	0.031
REACH 6	...	0.034	0.034	0.034	0.034	0.034
REACH 7	...	0.038	0.038	0.038	0.038	0.038
REACH 8	...	0.037	0.037	0.037	0.037	0.037
REACH 9	...	0.034	0.034	0.034	0.034	0.034
REACH 10	...	0.034	0.034	0.034	0.034	0.034
REACH 11	...	0.034	0.034	0.034	0.034	0.034
REACH 12	...	0.036	0.036	0.036	0.036	0.036
REACH 13	...	0.036	0.036	0.036	0.036	0.036

CROSS-SECTIONAL VARIABLES FOR OPT: 11
BELOW EXAMPLE-1

PARAMETER	UNITS	VARIABLE
**	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	2.000	0.000
2	2.000	0.000
3	0.500	0.000
4	0.500	-0.900
5	0.500	0.000
6	0.750	0.000
7	1.000	0.100
8	1.000	-0.500
9	1.000	0.000
10	1.100	0.000
11	1.000	0.000
12	1.000	0.000
13	1.400	0.000

DOWNSTREAM FLOW PARAMETERS FOR OPT: 11
BELOW EXAMPLE-1

PARAMETER	UNITS	VARIABLE	VALUE
***	********	********	*****
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

```
     EXAMPLE-2           OPT: 1


         1         0         0         5         3         0
1010011100
1937.     1156.     577.      216.      0.
5288.5    5228.5    5098.5    5038.5    5027.
 0.       5288.55   1.04      5027.     81.       1.43      5027.     0.
5288.5    5288.5    0.        0.        0.        0.        300.      13000.
0.        55.
13000.    13000.    13000.
0.        1.        55.
        12         5         6         4         0         0
         1         2         3         4        10        12
0.01      5047.
5027.     5037.     5051.     5107.     5125.
0.        590.      820.      1130.     1200.

5.01      4985.
4965.     4980.     5015.     5020.     5030.
0.        850.      1100.     1200.     1300.
0.        0.        3500.     4300.     5300.
8.51      4946.
4920.     4930.     4942.     4953.     4958.
0.        800.      4000.     11000.    15000.
0.        0.        0.        7000.     10000.
16.01     4830.
  4817.     4827.     4845.     4847.     4852.
  0.        884.      4000.     11000.    22000.
  0.        0.        30000.    27000.    25000.
22.51     4820.
  4805.     4812.     4814.     4825.     4830.
  0.        1000.     1200.     11000.    16000.
  0.        0.        0.        6000.     8000.
27.51     4800.
  4788.     4792.     4802.     4808.     4810.
  0.        286.      7000.     10000.     11000.
  0.        0.        0.        3500.     5000.
32.51      4777.
  4762.      4774.    4777.     4780.     4785.
  0.        352.      5000.     10000.     18000.
  0.        0.        9000.     16000.    24000.
37.51     4767.
  4752.     4763.     4768.     4773.     4778.
  0.        450.      3500.      6000.    9000.
  0.        0.        4000.     8500.     12000.
41.01     4756.
  4736.     4756.     4761.     4763.     4768.
  0.        540.      2000.     4000.     6000.
  0.        0.        3700.      3700.     5500.
43.01     4749.
  4729.     4737.     4749.     4757.     4759.
```

Appendix D.2.1-2

0.	250.	587.	1750.	2000.			
0.	0.	0.	1500.	2000.			
51.51	4674.						
4654.	4659.	4668.	4678.	4683.			
0.	70.	352.	400.	420.			
0.							
59.51	4612.						
4601.	4604.	4606.	4615.	4620.			
0.	245.	450.	500.	520.			
0.							
.08	.08	.08	.08	.08			
.06	.06	.06	.06	.06			
.031	.031	.031	.031	.031			
.034	.034	.034	.034	.034			
.038	.038	.038	.038	.038			
.037	.037	.037	.037	.037			
.034	.034	.034	.034	.034			
.034	.034	.034	.034	.034			
.034	.034	.034	.034	.034			
.036	.036	.036	.036	.036			
.036	.036	.036	.036	.036			
.5	.5	.5	.75	1.	1.	1.	1.
1.	1.1	1.4					
	-0.9			0.1	-0.5		
0.							
75000.	-.37	0.	0.				

```
                    PROGRAM DAMBRK---VERSION--6/20/88

                 ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
                      PRODUCED BY THE DAM BREAK OF
                               EXAMPLE-2
                                  ON
                                OPT: 1

                              ANALYSIS BY

                     BASED ON PROCEDURE DEVELOPED BY
               DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

                QUALITY CONTROL TESTING AND OTHER SUPPORT BY
                    JANICE M. LEWIS, RESEARCH HYDROLOGIST

                     HYDROLOGIC RESEARCH LABORATORY
                        W23, OFFICE OF HYDROLOGY
                     NOAA, NATIONAL WEATHER SERVICE
                     SILVER SPRING, MARYLAND  20910
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                    *******************************
                    ***                         ***
                    ***   SUMMARY OF INPUT DATA ***
                    ***                         ***
                    *******************************
                    *******************************


              INPUT CONTROL PARAMETERS FOR       EXAMPLE-2

PARAMETER                                                   VARIABLE   VALUE
*****************************************************       ********   *****

NUMBER OF DYNAMIC ROUTING REACHES                            KKN         1
TYPE OF RESERVOIR ROUTING                                    KUI         0
MULTIPLE DAM INDICATOR                                       MULDAM      0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY                      KDMP        5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS                    ITEH        3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9        NPRT        0
FLOOD-PLAIN MODEL PARAMETER                                  KFLP        0
METRIC INPUT/OUTPUT OPTION                                   METRIC      0

IOPUT= 1 0 1 0 0 1 1 1 0 0  0  0
```

EXAMPLE-2 RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (ACRES) SA(K)	ELEVATION (FT) HSA(K)
********************	***************
1937.0	5288.50
1156.0	5228.50
577.0	5098.50
216.0	5038.50
0.0	5027.00
0.0	0.00
0.0	0.00
0.0	0.00

1

EXAMPLE-2 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
**	*****	********	********
LENGTH OF RESERVOIR	MILE	RLM	0.00
ELEVATION OF WATER SURFACE	FEET	YO	5288.55
SIDE SLOPE OF BREACH		Z	1.04
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	5027.00
WIDTH OF BASE OF BREACH	FEET	BB	81.00
TIME TO MAXIMUM BREACH SIZE	HOUR	TFH	1.43
ELEVATION (MSL) OF BOTTOM OF DAM	FEET	DATUM	5027.00
VOLUME-SURFACE AREA PARAMETER		VOL	0.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	5288.50
ELEVATION OF TOP OF DAM	FEET	HD	5288.50
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	300.00
DISCHARGE THRU TURBINES	CFS	QT	13000.00

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 55.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-2
****

13000.00 13000.00 13000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 55.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPT: 1
BELOW EXAMPLE-2

PARAMETER	VARIABLE	VALUE
***	********	*******
NUMBER OF CROSS-SECTIONS	NS	12
MAXIMUM NUMBER OF TOP WIDTHS	NCS	5
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 3 4 10 12

CROSS-SECTIONAL VARIABLES FOR OPT: 1
BELOW EXAMPLE-2

PARAMETER	UNITS	VARIABLE
***	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)

Appendix D2.2-4

NUMBER OF CROSS-SECTION I
NUMBER OF ELEVATION LEVEL K
1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.010 FSTG(I) = 5047.00

HS ...	5027.0	5037.0	5051.0	5107.0	5125.0
BS ...	0.0	590.0	820.0	1130.0	1200.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 5.010 FSTG(I) = 4985.00

HS ...	4965.0	4980.0	5015.0	5020.0	5030.0
BS ...	0.0	850.0	1100.0	1200.0	1300.0
BSS ...	0.0	0.0	3500.0	4300.0	5300.0

CROSS-SECTION NUMBER 3
****

XS(I) = 8.510 FSTG(I) = 4946.00

HS ...	4920.0	4930.0	4942.0	4953.0	4958.0
BS ...	0.0	800.0	4000.0	11000.0	15000.0
BSS ...	0.0	0.0	0.0	7000.0	10000.0

CROSS-SECTION NUMBER 4
****

XS(I) = 16.010 FSTG(I) = 4830.00

HS ...	4817.0	4827.0	4845.0	4847.0	4852.0
BS ...	0.0	884.0	4000.0	11000.0	22000.0
BSS ...	0.0	0.0	30000.0	27000.0	25000.0

1

```
CROSS-SECTION NUMBER  5
***********************

XS(I) =     22.510   FSTG(I) = 4820.00

HS  ...    4805.0  4812.0  4814.0  4825.0  4830.0
BS  ...       0.0  1000.0  1200.0 11000.0 16000.0
BSS ...       0.0     0.0     0.0  6000.0  8000.0


CROSS-SECTION NUMBER  6
***********************

XS(I) =     27.510   FSTG(I) = 4800.00

HS  ...    4788.0  4792.0  4802.0  4808.0  4810.0
BS  ...       0.0   286.0  7000.0 10000.0 11000.0
BSS ...       0.0     0.0     0.0  3500.0  5000.0


CROSS-SECTION NUMBER  7
***********************

XS(I) =     32.510   FSTG(I) = 4777.00

HS  ...    4762.0  4774.0  4777.0  4780.0  4785.0
BS  ...       0.0   352.0  5000.0 10000.0 18000.0
BSS ...       0.0     0.0  9000.0 16000.0 24000.0


CROSS-SECTION NUMBER  8
***********************

XS(I) =     37.510   FSTG(I) = 4767.00

HS  ...    4752.0  4763.0  4768.0  4773.0  4778.0
BS  ...       0.0   450.0  3500.0  6000.0  9000.0
BSS ...       0.0     0.0  4000.0  8500.0 12000.0
1

CROSS-SECTION NUMBER  9
***********************

XS(I) =     41.010   FSTG(I) = 4756.00

HS  ...    4736.0  4756.0  4761.0  4763.0  4768.0
BS  ...       0.0   540.0  2000.0  4000.0  6000.0
BSS ...       0.0     0.0  3700.0  3700.0  5500.0
```

CROSS-SECTION NUMBER 10
****

XS(I) = 43.010 FSTG(I) = 4749.00

HS ...	4729.0	4737.0	4749.0	4757.0	4759.0
BS ...	0.0	250.0	587.0	1750.0	2000.0
BSS ...	0.0	0.0	0.0	1500.0	2000.0

CROSS-SECTION NUMBER 11
****

XS(I) = 51.510 FSTG(I) = 4674.00

HS ...	4654.0	4659.0	4668.0	4678.0	4683.0
BS ...	0.0	70.0	352.0	400.0	420.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 12
****

XS(I) = 59.510 FSTG(I) = 4612.00

HS ...	4601.0	4604.0	4606.0	4615.0	4620.0
BS ...	0.0	245.0	450.0	500.0	520.0
BSS ...	0.0	0.0	0.0	0.0	0.0

1

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	0.080	0.080	0.080	0.080	0.080
REACH 2 ...	0.060	0.060	0.060	0.060	0.060
REACH 3 ...	0.031	0.031	0.031	0.031	0.031
REACH 4 ...	0.034	0.034	0.034	0.034	0.034
REACH 5 ...	0.038	0.038	0.038	0.038	0.038
REACH 6 ...	0.037	0.037	0.037	0.037	0.037
REACH 7 ...	0.034	0.034	0.034	0.034	0.034
REACH 8 ...	0.034	0.034	0.034	0.034	0.034
REACH 9 ...	0.034	0.034	0.034	0.034	0.034
REACH 10 ...	0.036	0.036	0.036	0.036	0.036
REACH 11 ...	0.036	0.036	0.036	0.036	0.036

1

CROSS-SECTIONAL VARIABLES FOR OPT: 1
BELOW EXAMPLE-2

PARAMETER	UNITS	VARIABLE
***	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	0.500	0.000
2	0.500	-0.900
3	0.500	0.000
4	0.750	0.000
5	1.000	0.100
6	1.000	-0.500
7	1.000	0.000
8	1.000	0.000
9	1.000	0.000
10	1.100	0.000
11	1.400	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPT: 1
BELOW EXAMPLE-2

PARAMETER	UNITS	VARIABLE	VALUE
***	*****	********	*******
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	75000.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	-0.370
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

EXAMPLE-3 OPT: 12/DAM/BRIDGE

```
        1         1         2         3         4
        1         5
1500.     0.
1050.     1000.
0.        1050.     0.        1000.     100.      4.0       1000.     0.
1050.     1050.     0.        0.        0.        0.        0.        5000.
0.        0.        0.        0.        0.         1.        980.      0.
1100.     1005.     1000.     0.        0.        50.       0.8
980.      990.      1002.     1002.1
0.        300.      300.      0.
0.        15.
3000.     3000.     3000.     3000.
0.        24.       48.       72.
        9         3         5         4         0         0
        1         2         5         6         9
0.0
1000.02   1010.     1025.
0.        500.      1000.
0.
0.01
1000.     1010.     1025.
0.        500.      1000.
0.
5.
990.      1000.     1015.
0.        500.      1000.
0.
9.
982.      992.      1007.
0.        500.      1000.
0.
10.
980.      990.      1005.
0.        300.      300.
0.        200.      700.
10.01
979.98    990.      1005.
0.        300.      300.
0.        200.      300.
11.
978.      988.      1003.
0.        500.      1000.
0.
15.
970.      980.      995.
0.        500.      1000.
0.
20.
960.      970.      985.
```

0.	500.	1000.					
0.							
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.06	0.06	0.06	0.06				
0.5	0.5	0.5	0.2	0.5	0.2	0.5	0.5
0.	0.	0.	0.2	0.	-1.0	0.	0.
0.	0.	0.2	0.				

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-3
ON
OPT: 12/DAM/BRIDGE

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
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INPUT CONTROL PARAMETERS FOR EXAMPLE-3

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	1
MULTIPLE DAM INDICATOR	MULDAM	2
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	4
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

IDAM= 1
IDAM= 5

EXAMPLE-3 RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (ACRES) SA(K)	ELEVATION (FT) HSA(K)
1500.0	1050.00
0.0	1000.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00

DAM NUMBER 1

EXAMPLE-3 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
ELEVATION OF WATER SURFACE	FEET	YO	1050.00
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	1000.00
WIDTH OF BASE OF BREACH	FEET	BB	100.00
TIME TO MAXIMUM BREACH SIZE	HR	TFH	4.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	1050.00
ELEVATION OF TOP OF DAM	FEET	HD	1050.00
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	0.00
DISCHARGE THRU TURBINES	CFS	QT	5000.00

CDO SHOULD NOT BE 0.0 IF OVERTOPPING MAY OCCUR

BRIDGE NUMBER 1

SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	0.00
WIDTH OF BASE OF BREACH	FEET	BB	0.00
TIME TO MAXIMUM BREACH SIZE	HR	TFH	1.00

BRIDGE NUMBER 1

ELEVATION OF WATER WHEN BREACHED	FEET	HF	1100.00
ELEVATION OF CREST OF UPPER ROAD EMBANKMENT	FEET	HD	1005.00
LENGTH OF CREST OF UPPER EMBANKMENT	FEET	HSPD	1000.00
ELEVATION OF CREST OF LOWER ROAD EMBANKMENT	FEET	HGTD	0.00
LENGTH OF CREST OF LOWER EMBANKMENT	FEET	CSD	0.00
WIDTH OF CREST OF ROAD EMBANKMENT	FEET	CGD	50.00
BRIDGE DISCHARGE COEFFICIENT		CDOD	0.80
TIME STEP FOR EMBANKMENT OVERTOPPING	HR	QT	0.00

HSBR(L,K):

980.	990.	1002.	1002.	0.	0.	0.	0.

BSBR(L,K):

0.	300.	300.	0.	0.	0.	0.	0.

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 15.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-3
****

3000.00 3000.00 3000.00 3000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 24.0000 48.0000 72.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPT: 12/DAM/BRIDGE
BELOW EXAMPLE-3

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF CROSS-SECTIONS	NS	9
MAXIMUM NUMBER OF TOP WIDTHS	NCS	3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	5
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 5 6 9

CROSS-SECTIONAL VARIABLES FOR OPT: 12/DAM/BRIDGE
BELOW EXAMPLE-3

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	1000.0	1010.0	1025.0
BS ...	0.0	500.0	1000.0
BSS ...	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 0.010 FSTG(I) = 0.00

HS ...	1000.0	1010.0	1025.0
BS ...	0.0	500.0	1000.0
BSS ...	0.0	0.0	0.0

```
CROSS-SECTION NUMBER  3
***********************

XS(I) =     5.000   FSTG(I) =    0.00

HS  ...     990.0  1000.0  1015.0
BS  ...       0.0   500.0  1000.0
BSS ...       0.0     0.0     0.0


CROSS-SECTION NUMBER  4
***********************

XS(I) =     9.000   FSTG(I) =    0.00

HS  ...     982.0   992.0  1007.0
BS  ...       0.0   500.0  1000.0
BSS ...       0.0     0.0     0.0
1


CROSS-SECTION NUMBER  5
***********************

XS(I) =    10.000   FSTG(I) =    0.00

HS  ...     980.0   990.0  1005.0
BS  ...       0.0   300.0   300.0
BSS ...       0.0   200.0   700.0


CROSS-SECTION NUMBER  6
***********************

XS(I) =    10.010   FSTG(I) =    0.00

HS  ...     980.0   990.0  1005.0
BS  ...       0.0   300.0   300.0
BSS ...       0.0   200.0   300.0


CROSS-SECTION NUMBER  7
***********************

XS(I) =    11.000   FSTG(I) =    0.00

HS  ...     978.0   988.0  1003.0
BS  ...       0.0   500.0  1000.0
BSS ...       0.0     0.0     0.0
```

```
CROSS-SECTION NUMBER  8
***********************

XS(I) =     15.000   FSTG(I) =    0.00

HS  ...     970.0    980.0   995.0
BS  ...       0.0    500.0  1000.0
BSS ...       0.0      0.0     0.0
1

CROSS-SECTION NUMBER  9
***********************

XS(I) =     20.000   FSTG(I) =    0.00

HS  ...     960.0    970.0   985.0
BS  ...       0.0    500.0  1000.0
BSS ...       0.0      0.0     0.0
1

          MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
                (CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
         *******************************************************

                   REACH  1 ... 0.060  0.060  0.060
                   REACH  2 ... 0.060  0.060  0.060
                   REACH  3 ... 0.060  0.060  0.060
                   REACH  4 ... 0.060  0.060  0.060
                   REACH  5 ... 0.060  0.060  0.060
                   REACH  6 ... 0.060  0.060  0.060
                   REACH  7 ... 0.060  0.060  0.060
                   REACH  8 ... 0.060  0.060  0.060
1

             CROSS-SECTIONAL VARIABLES FOR OPT: 12/DAM/BRIDGE
                          BELOW      EXAMPLE-3

PARAMETER                                       UNITS   VARIABLE
********************************************    *****   ********

MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN     MILE     DXM(I)
     CROSS-SECTIONS
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN             FKC(I)
     CROSS-SECTIONS
```

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	0.500	0.000
2	0.500	0.000
3	0.500	0.000
4	0.200	0.200
5	0.500	0.000
6	0.200	-1.000
7	0.500	0.000
8	0.500	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPT: 12/DAM/BRIDGE
BELOW EXAMPLE-3

PARAMETER	UNITS	VARIABLE	VALUE
**	********	********	******
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.2000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

```
    EXAMPLE-4          OPT:11/LP/GAT/CONV

        1         1         1         3         4         0         1
        1
1500.     0.
1050.     1000.
0.        1049.9    0.        1000.     100.      0.5       1000.     0.
1050.     1050.     1045.     1040.     60.       28.8      3000.     0.
0.        15.
3000.     13000.    3000.     3000.
0.        1.        20.       50.
        6         3         6         4         0         0         0         6
        1         2         3         4         5         6
0.
1000.     1010.     1050.
0.        50.       100.
0.        225.      450.
0.        225.      450.
0.
0.01
1000.     1010.      1025.
0.        50.        100.
0.        225.       450.
0.        225.       450.
0.
5.
975.      985.      1000.
0.        75.       150.
0.        175.      500.
0.        100.      200.
0.
10.
950.      960.      975.
0.        200.      200.
0.        500.      1000.
0.        150.      250.
0.
15.
925.      935.      950.
0.        50.       100.
0.        100.      150.
0.        450.      800.
0.
20.
900.      910.      925.
0.        50.       150.
0.        75.       400.
0.        500.      600.
0.
0.04      0.04      0.04
0.06      0.06      0.06
```

Appendix D.4.1-2

```
0.06      0.06      0.06
0.04      0.05      0.04
0.06      0.06      0.07
0.06      0.08      0.09
0.04      0.05      0.04
0.06      0.06      0.05
0.05      0.06      0.07
0.04      0.035     0.04
0.06      0.06      0.06
0.05      0.06      0.05
0.04      0.04      0.03
0.06      0.07      0.09
0.06      0.07      0.07
1.0       1.0       1.0
1.5       1.5       1.1
1.5       1.5       1.1
1.4       1.4       1.0
1.3       1.3       1.0
2.0       0.13      .13       .13       .13
0.
0.        0.         0.        0.
        1
  28.8     28.8      43.2      57.6      28.8      28.8
  2.       2.        3.        4.        2.        2.
  0.0      0.5       1.0       1.5       10.       20.
```

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-4
ON
OPT:11/LP/GAT/CONV

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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```
******************************
******************************
***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE-4

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	1
MULTIPLE DAM INDICATOR	MULDAM	1
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	4
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	1
METRIC INPUT/OUTPUT OPTION	METRIC	0

IDAM= 1

EXAMPLE-4 RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (ACRES) SA(K)	ELEVATION (FT) HSA(K)
1500.0	1050.00
0.0	1000.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00

DAM NUMBER 1

EXAMPLE-4 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
ELEVATION OF WATER SURFACE	FEET	YO	1049.90
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	1000.00
WIDTH OF BASE OF BREACH	FEET	BB	100.00
TIME TO MAXIMUM BREACH SIZE	HR	TFH	0.50
ELEVATION OF WATER WHEN BREACHED	FEET	HF	1050.00
ELEVATION OF TOP OF DAM	FEET	HD	1050.00
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	1045.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	1040.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	60.00
DISCHARGE COEF. FOR GATE FLOW		CG	28.80
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	3000.00
DISCHARGE THRU TURBINES	CFS	QT	0.00

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 15.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-4
****

3000.00 13000.00 3000.00 3000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 20.0000 50.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPT:11/LP/GAT/CONV
BELOW EXAMPLE-4

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF CROSS-SECTIONS	NS	6
MAXIMUM NUMBER OF TOP WIDTHS	NCS	3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	6

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 3 4 5 6

CROSS-SECTIONAL VARIABLES FOR OPT:11/LP/GAT/CONV
BELOW EXAMPLE-4

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

Appendix D.4.2-4

```
CROSS-SECTION NUMBER  1
***********************

XS(I) =     0.000   FSTG(I) =    0.00

HS  ...    1000.0  1010.0  1050.0
BS  ...       0.0    50.0   100.0
BSL ...       0.0   225.0   450.0
BSR ...       0.0   225.0   450.0
BSS ...       0.0     0.0     0.0


CROSS-SECTION NUMBER  2
***********************

XS(I) =     0.010   FSTG(I) =    0.00

HS  ...    1000.0  1010.0  1025.0
BS  ...       0.0    50.0   100.0
BSL ...       0.0   225.0   450.0
BSR ...       0.0   225.0   450.0
BSS ...       0.0     0.0     0.0


CROSS-SECTION NUMBER  3
***********************

XS(I) =     5.000   FSTG(I) =    0.00

HS  ...     975.0   985.0  1000.0
BS  ...       0.0    75.0   150.0
BSL ...       0.0   175.0   500.0
BSR ...       0.0   100.0   200.0
BSS ...       0.0     0.0     0.0


CROSS-SECTION NUMBER  4
***********************

XS(I) =    10.000   FSTG(I) =    0.00

HS  ...     950.0   960.0   975.0
BS  ...       0.0   200.0   200.0
BSL ...       0.0   500.0  1000.0
BSR ...       0.0   150.0   250.0
BSS ...       0.0     0.0     0.0
1
```

```
CROSS-SECTION NUMBER  5
***********************

XS(I) =    15.000   FSTG(I) =    0.00

HS  ...    925.0   935.0   950.0
BS  ...      0.0    50.0   100.0
BSL ...      0.0   100.0   150.0
BSR ...      0.0   450.0   800.0
BSS ...      0.0     0.0     0.0


CROSS-SECTION NUMBER  6
***********************

XS(I) =    20.000   FSTG(I) =    0.00

HS  ...    900.0   910.0   925.0
BS  ...      0.0    50.0   150.0
BSL ...      0.0    75.0   400.0
BSR ...      0.0   500.0   600.0
BSS ...      0.0     0.0     0.0


     HS(1, 2) IS GREATER THAN HS(1, 1).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.
1

            MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
                   (CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
           ********************************************************

                        REACH  1 ... 0.040  0.040  0.040
                        REACH  1 ... 0.060  0.060  0.060
                        REACH  1 ... 0.060  0.060  0.060
                        REACH  2 ... 0.040  0.050  0.040
                        REACH  2 ... 0.060  0.060  0.070
                        REACH  2 ... 0.060  0.080  0.090
                        REACH  3 ... 0.040  0.050  0.040
                        REACH  3 ... 0.060  0.060  0.050
                        REACH  3 ... 0.050  0.060  0.070
                        REACH  4 ... 0.040  0.035  0.040
                        REACH  4 ... 0.060  0.060  0.060
                        REACH  4 ... 0.050  0.060  0.050
                        REACH  5 ... 0.040  0.040  0.030
                        REACH  5 ... 0.060  0.070  0.090
                        REACH  5 ... 0.060  0.070  0.070
```

SNC ...	1.00	1.00	1.00
SNC ...	1.50	1.50	1.10
SNC ...	1.50	1.50	1.10
SNC ...	1.40	1.40	1.00
SNC ...	1.30	1.30	1.00

1

CROSS-SECTIONAL VARIABLES FOR OPT:11/LP/GAT/CONV
BELOW EXAMPLE-4

PARAMETER	UNITS	VARIABLE
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	2.000	0.000
2	0.130	0.000
3	0.130	0.000
4	0.130	0.000
5	0.130	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPT:11/LP/GAT/CONV
BELOW EXAMPLE-4

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

(ICG(K),K=1,MULDAM) 1

GATE CONTROL CURVE FOR DAM NO. 1

GATE CONTROL COEFF.-CCCG(K,L)	GATE BOTTOM ELEV.(FT ABOVE OF SILL)-GBL(K,L)	TIME(HRS) OF GATE CONTROL COEFF.-TCG(K,L)
28.80	2.00	0.00
28.80	2.00	0.50
43.20	3.00	1.00
57.60	4.00	1.50
28.80	2.00	10.00
28.80	2.00	20.00

```
    EXAMPLE-5              SUB/SUPER/SUB

         9          0          0          3          5          0
   0.        10.
 100.       100.      12000.    100.       100.
   0.         1.        2.         3.        15.
        11          3          6          5          0          2
         1          3          5          6          9         11
   0.
   1000.     1010.     1100.
   0.        100.      1000.
   0.
   4.5
   977.5     987.5     1077.5
   0.        100.      1000.
   0.
   5.
   975.      985.      1075.
   0.        100.      1000.
   0.
   5.5
   977.5     987.5     1077.5
   0.        100.      1000.
   0.
   9.5
   997.5     1007.5    1097.5
   0.        100.      1000.
   0.
   10.
   1000.     1010.     1100.
   0.        100.      1000.
   0.
   10.5
   990.     1000.      1090.
   0.        100.      1000.
   0.
   14.5
   910.     920.      1010.
   0.       100.      1000.
   0.
   15.
   900.      910.      1000.
   0.        100.      1000.
   0.
   15.5
   897.5    907.5     997.5
   0.       100.      1000.
   0.
   20.
   875.0    885.      975.
   0.       100.      1000.
```

Appendix D.5.1-2

0.							
.060	.060	.060					
.060	.060	.060					
.060	.060	.060					
.060	.060	.060					
.01	.01	.01					
.010	.010	.010					
.010	.010	.010					
.010	.010	.010					
.060	.060	.060					
.060	.060	.060					
.5	.2	.2	.5	.2	.2	.5	.2
.2	.5						
0.	0.	0.	0.	0.	0.	0.	0.
0.	0.						
0.	0.	0.05	0.				

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-5
ON
SUB/SUPER/SUB

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE-5

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	9
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	5
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

```
DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) =     0.00  HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) =    10.0000  HRS.
BREX(BREACH EXPONENT) =  0.000
MUD(MUD FLOW OPTION) =  0
IWF(TYPE OF WAVE FRONT TRACKING) =   0
KPRES(WETTED PERIMETER OPTION) =  0
KSL(LANDSLIDE PARAMETER) =  0


                    INFLOW HYDROGRAPH TO       EXAMPLE-5
                   ***************************************

                100.00     100.00  12000.00    100.00     100.00


                    TIME OF INFLOW HYDROGRAPH ORDINATES

                0.0000     1.0000     2.0000     3.0000    15.0000
1


                CROSS-SECTIONAL PARAMETERS FOR   SUB/SUPER/SUB
                            BELOW        EXAMPLE-5

PARAMETER                                            VARIABLE   VALUE
********************************************         ********   *****

NUMBER OF CROSS-SECTIONS                               NS         11
MAXIMUM NUMBER OF TOP WIDTHS                           NCS         3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT          NTT         6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS             JNK         5
CROSS-SECTIONAL SMOOTHING PARAMETER                    KSA         0
DOWNSTREAM SUPERCRITICAL OR NOT                        KSUPC       2
NO. OF LATERAL INFLOW HYDROGRAPHS                      LQ          0
NO. OF POINTS IN GATE CONTROL CURVE                    KCG         0


               NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
                       (MAX NUMBER OF HYDROGRAPHS = 6)
               *************************************************

                         1    3    5    6    9   11
```

CROSS-SECTIONAL VARIABLES FOR SUB/SUPER/SUB
BELOW EXAMPLE-5

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	1000.0	1010.0	1100.0
BS ...	0.0	100.0	1000.0
BSS ...	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 4.500 FSTG(I) = 0.00

HS ...	977.5	987.5	1077.5
BS ...	0.0	100.0	1000.0
BSS ...	0.0	0.0	0.0

CROSS-SECTION NUMBER 3
****

XS(I) = 5.000 FSTG(I) = 0.00

HS ...	975.0	985.0	1075.0
BS ...	0.0	100.0	1000.0
BSS ...	0.0	0.0	0.0

```
CROSS-SECTION NUMBER  4
***********************

XS(I) =     5.500   FSTG(I) =    0.00

HS  ...    977.5   987.5  1077.5
BS  ...      0.0   100.0  1000.0
BSS ...      0.0     0.0     0.0
1

CROSS-SECTION NUMBER  5
***********************

XS(I) =     9.500   FSTG(I) =    0.00

HS  ...    997.5  1007.5  1097.5
BS  ...      0.0   100.0  1000.0
BSS ...      0.0     0.0     0.0

CROSS-SECTION NUMBER  6
***********************

XS(I) =    10.000   FSTG(I) =    0.00

HS  ...   1000.0  1010.0  1100.0
BS  ...      0.0   100.0  1000.0
BSS ...      0.0     0.0     0.0

CROSS-SECTION NUMBER  7
***********************

XS(I) =    10.500   FSTG(I) =    0.00

HS  ...    990.0  1000.0  1090.0
BS  ...      0.0   100.0  1000.0
BSS ...      0.0     0.0     0.0

CROSS-SECTION NUMBER  8
***********************

XS(I) =    14.500   FSTG(I) =    0.00

HS  ...    910.0   920.0  1010.0
BS  ...      0.0   100.0  1000.0
BSS ...      0.0     0.0     0.0
1
```

```
CROSS-SECTION NUMBER  9
***********************

XS(I) =     15.000   FSTG(I) =    0.00

HS  ...     900.0    910.0  1000.0
BS  ...       0.0    100.0  1000.0
BSS ...       0.0      0.0     0.0


CROSS-SECTION NUMBER 10
***********************

XS(I) =     15.500   FSTG(I) =    0.00

HS  ...     897.5    907.5   997.5
BS  ...       0.0    100.0  1000.0
BSS ...       0.0      0.0     0.0


CROSS-SECTION NUMBER 11
***********************

XS(I) =     20.000   FSTG(I) =    0.00

HS  ...     875.0    885.0   975.0
BS  ...       0.0    100.0  1000.0
BSS ...       0.0      0.0     0.0


     HS(1, 4) IS GREATER THAN HS(1, 3).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.


     HS(1, 5) IS GREATER THAN HS(1, 4).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.


     HS(1, 6) IS GREATER THAN HS(1, 5).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.
1
```

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1	...	0.060	0.060	0.060
REACH 2	...	0.060	0.060	0.060
REACH 3	...	0.060	0.060	0.060
REACH 4	...	0.060	0.060	0.060
REACH 5	...	0.010	0.010	0.010
REACH 6	...	0.010	0.010	0.010
REACH 7	...	0.010	0.010	0.010
REACH 8	...	0.010	0.010	0.010
REACH 9	...	0.060	0.060	0.060
REACH 10	...	0.060	0.060	0.060

1

CROSS-SECTIONAL VARIABLES FOR SUB/SUPER/SUB
BELOW EXAMPLE-5

PARAMETER	UNITS	VARIABLE
***************************************	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	0.500	0.000
2	0.200	0.000
3	0.200	0.000
4	0.500	0.000
5	0.200	0.000
6	0.200	0.000
7	0.500	0.000
8	0.200	0.000
9	0.200	0.000
10	0.500	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR SUB/SUPER/SUB
BELOW EXAMPLE-5

PARAMETER	UNITS	VARIABLE	VALUE
***	********	********	******
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0500
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

```
    EXAMPLE-6              OPT:11/LP/CONV/PRES

          1          1          1          5          4          6          0
        101        102        103        151        152        153
1010001100
          1
 1500.     0.
 1050.     1000.
 0.        1049.9     0.         1000.      25.        0.5        1000.      0.
 1050.     1050.      1045.      0.         60.        0.         3000.      0.
 0.         9.                              0          0          1
 3000.     13000.     3000.      3000.
 0.        1.         20.        50.
          8          4          6          9          0          0          0
          1          2          3          4          5          6
 0.
 1000.     1010.      1050.      1060.
 50.       50.        1000.      1000.
 0.
 0.01
 1000.     1010.       1025.      1050.
 50.       50.         150.       150.
 0.
 5.
 975.      985.       1000.       1025.
 50.       50.        150.        150.
 0.
 10.
 950.      960.       975.        1000.
 50.       50.        150.        150.
 0.
 10.1
 950.      960.       962.1       990.
 50.       50.        0.01        0.01
 0.
 14.9
 925.      935.       937.1       965.
 50.       50.        0.01        0.01
 0.
 15.
 925.      935.       950.        975.
 50.       50.        150.        150.
 0.
 20.
 900.      910.       925.        950.
 50.       50.        150.        150.
 0.
 0.04      0.04       0.04        0.04
 0.04      0.05       0.04        0.04
 0.04      0.05       0.04        0.04
 0.04      0.035      0.04        0.04
```

Appendix D.6.1-2

0.04	0.04	0.04	0.04			
0.04	0.04	0.04	0.04			
0.04	0.04	0.04	0.04			
2.0	0.10	0.10	0.05	0.10	0.05	0.12
0.						
0.	0.	0.	0.			

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-6
ON
OPT:11/LP/CONV/PRES

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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******************************
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***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE-6

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	1
MULTIPLE DAM INDICATOR	MULDAM	1
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	4
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	6
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

(NPT(K),K=1,NPRT)   101   102   103   151   152   153

IOPUT= 1 0 1 0 0 0 1 1 0 0 0 0

IDAM= 1

Appendix D.6.2-2

EXAMPLE-6 RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (ACRES) SA(K)	ELEVATION (FT) HSA(K)
1500.0	1050.00
0.0	1000.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00

DAM NUMBER 1

EXAMPLE-6 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
ELEVATION OF WATER SURFACE	FEET	YO	1049.90
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	1000.00
WIDTH OF BASE OF BREACH	FEET	BB	25.00
TIME TO MAXIMUM BREACH SIZE	HR	TFH	0.50
ELEVATION OF WATER WHEN BREACHED	FEET	HF	1050.00
ELEVATION OF TOP OF DAM	FEET	HD	1050.00
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	1045.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	60.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	3000.00
DISCHARGE THRU TURBINES	CFS	QT	0.00

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 9.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 1
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-6
****

3000.00 13000.00 3000.00 3000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 20.0000 50.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPT:11/LP/CONV/PRES
BELOW EXAMPLE-6

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF CROSS-SECTIONS	NS	8
MAXIMUM NUMBER OF TOP WIDTHS	NCS	4
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	9
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 3 4 5 6

CROSS-SECTIONAL VARIABLES FOR OPT:11/LP/CONV/PRES
BELOW EXAMPLE-6

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

Appendix D.6.2-4

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	1000.0	1010.0	1050.0	1060.0
BS ...	50.0	50.0	1000.0	1000.0
BSS ...	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 0.010 FSTG(I) = 0.00

HS ...	1000.0	1010.0	1025.0	1050.0
BS ...	50.0	50.0	150.0	150.0
BSS ...	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 3
****

XS(I) = 5.000 FSTG(I) = 0.00

HS ...	975.0	985.0	1000.0	1025.0
BS ...	50.0	50.0	150.0	150.0
BSS ...	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 4
****

XS(I) = 10.000 FSTG(I) = 0.00

HS ...	950.0	960.0	975.0	1000.0
BS ...	50.0	50.0	150.0	150.0
BSS ...	0.0	0.0	0.0	0.0

1

CROSS-SECTION NUMBER 5
****

XS(I) = 10.100 FSTG(I) = 0.00

HS ...	950.0	960.0	962.1	990.0
BS ...	50.0	50.0	0.0	0.0
BSS ...	0.0	0.0	0.0	0.0

```
CROSS-SECTION NUMBER  6
***********************

XS(I) =    14.900   FSTG(I) =    0.00

HS  ...    925.0   935.0   937.1   965.0
BS  ...     50.0    50.0     0.0     0.0
BSS ...      0.0     0.0     0.0     0.0


CROSS-SECTION NUMBER  7
***********************

XS(I) =    15.000   FSTG(I) =    0.00

HS  ...    925.0   935.0   950.0   975.0
BS  ...     50.0    50.0   150.0   150.0
BSS ...      0.0     0.0     0.0     0.0


CROSS-SECTION NUMBER  8
***********************

XS(I) =    20.000   FSTG(I) =    0.00

HS  ...    900.0   910.0   925.0   950.0
BS  ...     50.0    50.0   150.0   150.0
BSS ...      0.0     0.0     0.0     0.0

     HS(1, 2) IS GREATER THAN HS(1, 1).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.


     HS(1, 5) IS GREATER THAN HS(1, 4).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.


     HS(1, 7) IS GREATER THAN HS(1, 6).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.
1
```

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1	...	0.040	0.040	0.040	0.040
REACH 2	...	0.040	0.050	0.040	0.040
REACH 3	...	0.040	0.050	0.040	0.040
REACH 4	...	0.040	0.035	0.040	0.040
REACH 5	...	0.040	0.040	0.040	0.040
REACH 6	...	0.040	0.040	0.040	0.040
REACH 7	...	0.040	0.040	0.040	0.040

1

CROSS-SECTIONAL VARIABLES FOR OPT:11/LP/CONV/PRES
BELOW EXAMPLE-6

PARAMETER	UNITS	VARIABLE
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	2.000	0.000
2	0.100	0.000
3	0.100	0.000
4	0.050	0.000
5	0.100	0.000
6	0.050	0.000
7	0.120	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPT:11/LP/CONV/PRES
BELOW EXAMPLE-6

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000

```
    EXAMPLE 7:          4 FLDPLAIN COMPART

        9         0         0         3         4
0.         30.
500.       20000.   500.       500.
0.         12.      24.        30.
        6         2         6         9         0         0         0        -4
        1         2         3         4         5         6
0.
100.      200.
500.      500.
0.
0.5
99.5      199.5
500.      500.
0.
1.0
99.0      199.0
500.      500.
0.
1.5
98.5      198.5
500.      500.
0.
2.0
98.0      198.0
500.      500.
0.
2.5
97.5      197.5
500.      500.
0.
0.04      0.04
0.04      0.04
0.04      0.04
0.04      0.04
0.04      0.04
0.5       0.5        0.20      0.5       0.5
0.
0.        0.         0.2       0.
        2
        3         4         0  105.0     3.0       100.
1000.     20000.
100.      120.
0.        0.         0.        0.
300.      3000.
105.5      120.
        4         5         0  104.      3.0        99.
1000.     30000.
99.       120.
0.        0.         0.        0.
```

```
0.        0.
0.        0.
        3         4         0 104.5    3.0       100.
1000.     25000.
100.      120.
0.        0.        0.        0.
100.      2500.
105.5     120.
        4         5         0 103.      3.0         99.
1000.     35000.
99.       120.
0.        0.        0.        0.
0.        0.
0.        0.
        0
```

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE 7:
ON
4 FLDPLAIN COMPART

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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******************************
******************************
***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE 7:

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF DYNAMIC ROUTING REACHES	KKN	9
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	4
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

```
DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) =     0.00  HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) =    30.0000  HRS.
BREX(BREACH EXPONENT) =  0.000
MUD(MUD FLOW OPTION) =  0
IWF(TYPE OF WAVE FRONT TRACKING) =   0
KPRES(WETTED PERIMETER OPTION) =  0
KSL(LANDSLIDE PARAMETER) =  0


                     INFLOW HYDROGRAPH TO    EXAMPLE 7:
                    ************************************

                    500.00  20000.00    500.00    500.00


                     TIME OF INFLOW HYDROGRAPH ORDINATES

                    0.0000    12.0000   24.0000   30.0000
1


             CROSS-SECTIONAL PARAMETERS FOR  4 FLDPLAIN COMPART
                            BELOW    EXAMPLE 7:

PARAMETER                                        VARIABLE   VALUE
********************************************     ********   *****

NUMBER OF CROSS-SECTIONS                           NS          6
MAXIMUM NUMBER OF TOP WIDTHS                       NCS         2
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT      NTT         6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS         JNK         9
CROSS-SECTIONAL SMOOTHING PARAMETER                KSA         0
DOWNSTREAM SUPERCRITICAL OR NOT                    KSUPC       0
NO. OF LATERAL INFLOW HYDROGRAPHS                  LQ          0
NO. OF POINTS IN GATE CONTROL CURVE                KCG        -4


              NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
                       (MAX NUMBER OF HYDROGRAPHS = 6)
             **************************************************

                          1    2    3    4    5    6
```

CROSS-SECTIONAL VARIABLES FOR 4 FLDPLAIN COMPART
BELOW EXAMPLE 7:

PARAMETER	UNITS	VARIABLE
***	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	100.0	200.0
BS ...	500.0	500.0
BSS ...	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 0.500 FSTG(I) = 0.00

HS ...	99.5	199.5
BS ...	500.0	500.0
BSS ...	0.0	0.0

CROSS-SECTION NUMBER 3
****

XS(I) = 1.000 FSTG(I) = 0.00

HS ...	99.0	199.0
BS ...	500.0	500.0
BSS ...	0.0	0.0

```
CROSS-SECTION NUMBER  4
***********************

XS(I) =     1.500   FSTG(I) =    0.00

HS  ...     98.5   198.5
BS  ...    500.0   500.0
BSS ...      0.0     0.0
1


CROSS-SECTION NUMBER  5
***********************

XS(I) =     2.000   FSTG(I) =    0.00

HS  ...     98.0   198.0
BS  ...    500.0   500.0
BSS ...      0.0     0.0


CROSS-SECTION NUMBER  6
***********************

XS(I) =     2.500   FSTG(I) =    0.00

HS  ...     97.5   197.5
BS  ...    500.0   500.0
BSS ...      0.0     0.0
1


            MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
                   (CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
            *******************************************************

                           REACH  1 ... 0.040  0.040
                           REACH  2 ... 0.040  0.040
                           REACH  3 ... 0.040  0.040
                           REACH  4 ... 0.040  0.040
                           REACH  5 ... 0.040  0.040

1
```

CROSS-SECTIONAL VARIABLES FOR 4 FLDPLAIN COMPART
BELOW EXAMPLE 7:

PARAMETER	UNITS	VARIABLE
**	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	0.500	0.000
2	0.500	0.000
3	0.200	0.000
4	0.500	0.000
5	0.500	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR 4 FLDPLAIN COMPART
BELOW EXAMPLE 7:

PARAMETER	UNITS	VARIABLE	VALUE
***	********	********	******
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.2000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

DXM(I)= 0.500 0.500 0.200 0.500 0.500

NPLD= 2

NXPI(K) = 3 NXPN(K) = 4 NQLP(K) = 0 PWELV(K) = 105.00 PCWR(K) = 3.00 PEO(K) = 100.00 QMINP(K) = 0.00

(PSA(I,K),I=1,8)

1000.00	20000.00	0.00	0.00	0.00	0.00	0.00	0.00

(PEL(I,K),I=1,8)

100.00	120.00	0.00	0.00	0.00	0.00	0.00	0.00

```
     (QPU(I,K),I=1,ITEH)
0.00       0.00       0.00       0.00

     (COFF(I,K),I=1,8)
300.00   3000.00      0.00       0.00       0.00       0.00       0.00       0.00

     (HCFF(I,K),I=1,8)
105.50    120.00      0.00       0.00       0.00       0.00       0.00       0.00

NXPI(K) = 4  NXPN(K) = 5  NQLP(K) = 0  PWELV(K) = 104.00  PCWR(K) = 3.00  PEO(K) = 99.00  QMINP(K) = 0.00

     (PSA(I,K),I=1,8)
1000.00  30000.00     0.00       0.00       0.00       0.00       0.00       0.00

     (PEL(I,K),I=1,8)
99.00     120.00      0.00       0.00       0.00       0.00       0.00       0.00

     (QPU(I,K),I=1,ITEH)
0.00       0.00       0.00       0.00

     (COFF(I,K),I=1,8)
0.00       0.00       0.00       0.00       0.00       0.00       0.00       0.00

     (HCFF(I,K),I=1,8)
0.00       0.00       0.00       0.00       0.00       0.00       0.00       0.00

NXPI(K) = 3  NXPN(K) = 4  NQLP(K) = 0  PWELV(K) = 104.50  PCWR(K) = 3.00  PEO(K) = 100.00  QMINP(K) = 0.00

     (PSA(I,K),I=1,8)
1000.00  25000.00     0.00       0.00       0.00       0.00       0.00       0.00

     (PEL(I,K),I=1,8)
100.00    120.00      0.00       0.00       0.00       0.00       0.00       0.00

     (QPU(I,K),I=1,ITEH)
0.00       0.00       0.00       0.00

     (COFF(I,K),I=1,8)
100.00   2500.00      0.00       0.00       0.00       0.00       0.00       0.00

     (HCFF(I,K),I=1,8)
105.50    120.00      0.00       0.00       0.00       0.00       0.00       0.00

NXPI(K) = 4  NXPN(K) = 5  NQLP(K) = 0  PWELV(K) = 103.00  PCWR(K) = 3.00  PEO(K) = 99.00  QMINP(K) = 0.00
```

```
     (PSA(I,K),I=1,8)
1000.00  35000.00      0.00       0.00       0.00       0.00       0.00       0.00

     (PEL(I,K),I=1,8)
99.00    120.00       0.00       0.00       0.00       0.00       0.00       0.00

     (QPU(I,K),I=1,ITEH)
0.00       0.00       0.00       0.00

     (COFF(I,K),I=1,8)
0.00       0.00       0.00       0.00       0.00       0.00       0.00       0.00

     (HCFF(I,K),I=1,8)
0.00       0.00       0.00       0.00       0.00       0.00       0.00       0.00

NPM=  0
```

EXAMPLE-8 OPT: 1/METRIC

1	0	0	5	3	0	0	1
1010011100							
7.8389	4.6783	2.3351	0.8741	0.0			
1611.86	1593.57	1553.95	1535.66	1532.15			
0.	1611.86	1.04	1532.15	24.6876	1.43	1532.15	
1611.86	1611.86	0.	0.	0.	0.	50.488	368.06
0.	55.	0.		0	0	0	0
368.06	368.06	368.06					
0.	1.	55.					
12	5	6	4	0	0		
1	2	3	4	10	12		
0.016	1538.25						
1532.15	1535.20	1539.47	1556.53	1562.02			
0.	179.82	249.92	344.41	365.74			
0.	0.	0.	0.	0.			
8.062	1519.35						
1513.26	1517.83	1528.50	1530.02	1533.07			
0.	259.07	335.26	365.74	396.22			
0.	0.	1066.75	1310.58	1615.36			
13.695	1507.47						
1499.54	1502.59	1506.25	1509.60	1511.12			
0.	243.83	1219.14	3352.64	4571.78			
0.	0.	0.	2133.50	3047.85			
25.76	1472.11						
1468.15	1471.20	1476.68	1477.69	1478.82			
0.	269.43	1219.14	3352.64	6705.27			
0.	0.	9143.55	8229.20	7619.63			
36.22	1469.06						
1464.99	1466.62	1467.24	1470.59	1472.11			
0.	304.79	365.74	3352.64	4876.56			
0.	0.	0.	1828.71	2438.28			
44.27	1462.97						
1459.31	1460.53	1463.58	1465.41	1466.02			
0.	87.17	2133.50	3047.85	3352.64			
0.	0.	0.	1066.75	1523.93			
52.32	1455.96						
1451.39	1455.04	1455.96	1456.87	1458.40			
0.	107.28	1523.93	3047.85	5486.13			
0.	0.	2743.07	4876.56	7314.84			
60.36	1452.91						
1448.34	1451.69	1453.22	1454.74	1456.26			
0.	137.15	1066.75	1828.71	2743.07			
0.	0.	1219.14	2590.67	3657.42			
66.00	1449.56						
1443.46	1449.56	1451.08	1451.69	1453.22			
0.	164.58	609.57	1219.14	1828.71			
0.	0.	1127.70	1127.70	1676.32			
69.215	1447.42						
1441.33	1443.77	1447.42	1449.86	1450.47			

0.	76.20	178.91	533.37	609.57			
0.	0.	0.	457.18	609.57			
82.89	1424.56						
1418.47	1419.99	1422.74	1425.78	1427.31			
0.	21.33	107.28	121.91	128.01			
0.							
95.77	1405.67						
1402.32	1403.23	1403.84	1406.58	1408.11			
0.	74.67	137.15	152.39	158.48			
0.							
.08	.08	.08	.08	.08			
.06	.06	.06	.06	.06			
.031	.031	.031	.031	.031			
.034	.034	.034	.034	.034			
.038	.038	.038	.038	.038			
.037	.037	.037	.037	.037			
.034	.034	.034	.034	.034			
.034	.034	.034	.034	.034			
.034	.034	.034	.034	.034			
.036	.036	.036	.036	.036			
.036	.036	.036	.036	.036			
.8096	.8096	.8096	1.207	1.609	1.609	1.609	1.609
1.609	1.770	2.253					
	-0.9			0.1	-0.5		
0.							
2123.4	-0.0344						

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-8
ON
OPT: 1/METRIC

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
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INPUT CONTROL PARAMETERS FOR EXAMPLE-8

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	1

IOPUT= 1 0 1 0 0 1 1 1 0 0 0 0

EXAMPLE-8 RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (SQ KM) SA(K)	ELEVATION (M) HSA(K)
********************	***************
7.839	1611.86
4.678	1593.57
2.335	1553.95
0.874	1535.66
0.0	1532.15
0.0	0.00
0.0	0.00
0.0	0.00

1

EXAMPLE-8 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
**	*******	******	*********
LENGTH OF RESERVOIR	KM	RLM	0.00
ELEVATION OF WATER SURFACE	M	YO	1611.86
SIDE SLOPE OF BREACH		Z	1.04
ELEVATION OF BOTTOM OF BREACH	M	YBMIN	1532.15
WIDTH OF BASE OF BREACH	M	BB	24.69
TIME TO MAXIMUM BREACH SIZE	HOUR	TFH	1.43
ELEVATION (MSL) OF BOTTOM OF DAM	M	DATUM	1532.15
VOLUME-SURFACE AREA PARAMETER		VOL	0.00
ELEVATION OF WATER WHEN BREACHED	M	HF	1611.86
ELEVATION OF TOP OF DAM	M	HD	1611.86
ELEVATION OF UNCONTROLLED SPILLWAY CREST	M	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	M	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	50.49
DISCHARGE THRU TURBINES	CMS	QT	368.06

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 55.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-8
****

368.06 368.06 368.06

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 55.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPT: 1/METRIC
BELOW EXAMPLE-8

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF CROSS-SECTIONS	NS	12
MAXIMUM NUMBER OF TOP WIDTHS	NCS	5
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 3 4 10 12

CROSS-SECTIONAL VARIABLES FOR OPT: 1/METRIC
BELOW EXAMPLE-8

PARAMETER	UNITS	VARIABLE
***	*****	********
LOCATION OF CROSS-SECTION	KM	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	M	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	M	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	M	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	M	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

Appendix D.8.2-4

CROSS-SECTION NUMBER 1
****

XS(I) = 0.016 FSTG(I) = 1538.25

HS ...	1532.1	1535.2	1539.5	1556.5	1562.0
BS ...	0.0	179.8	249.9	344.4	365.7
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 8.062 FSTG(I) = 1519.35

HS ...	1513.3	1517.8	1528.5	1530.0	1533.1
BS ...	0.0	259.1	335.3	365.7	396.2
BSS ...	0.0	0.0	1066.7	1310.6	1615.4

CROSS-SECTION NUMBER 3
****

XS(I) = 13.695 FSTG(I) = 1507.47

HS ...	1499.5	1502.6	1506.2	1509.6	1511.1
BS ...	0.0	243.8	1219.1	3352.6	4571.8
BSS ...	0.0	0.0	0.0	2133.5	3047.9

CROSS-SECTION NUMBER 4
****

XS(I) = 25.760 FSTG(I) = 1472.11

HS ...	1468.1	1471.2	1476.7	1477.7	1478.8
BS ...	0.0	269.4	1219.1	3352.6	6705.3
BSS ...	0.0	0.0	9143.6	8229.2	7619.6

1

CROSS-SECTION NUMBER 5
****

XS(I) = 36.220 FSTG(I) = 1469.06

HS ...	1465.0	1466.6	1467.2	1470.6	1472.1
BS ...	0.0	304.8	365.7	3352.6	4876.6
BSS ...	0.0	0.0	0.0	1828.7	2438.3

CROSS-SECTION NUMBER 6
****

XS(I) = 44.270 FSTG(I) = 1462.97

HS ...	1459.3	1460.5	1463.6	1465.4	1466.0
BS ...	0.0	87.2	2133.5	3047.9	3352.6
BSS ...	0.0	0.0	0.0	1066.7	1523.9

CROSS-SECTION NUMBER 7
****

XS(I) = 52.320 FSTG(I) = 1455.96

HS ...	1451.4	1455.0	1456.0	1456.9	1458.4
BS ...	0.0	107.3	1523.9	3047.9	5486.1
BSS ...	0.0	0.0	2743.1	4876.6	7314.8

CROSS-SECTION NUMBER 8
****

XS(I) = 60.360 FSTG(I) = 1452.91

HS ...	1448.3	1451.7	1453.2	1454.7	1456.3
BS ...	0.0	137.1	1066.7	1828.7	2743.1
BSS ...	0.0	0.0	1219.1	2590.7	3657.4

1

CROSS-SECTION NUMBER 9
****

XS(I) = 66.000 FSTG(I) = 1449.56

HS ...	1443.5	1449.6	1451.1	1451.7	1453.2
BS ...	0.0	164.6	609.6	1219.1	1828.7
BSS ...	0.0	0.0	1127.7	1127.7	1676.3

CROSS-SECTION NUMBER 10
****

XS(I) = 69.215 FSTG(I) = 1447.42

HS ...	1441.3	1443.8	1447.4	1449.9	1450.5
BS ...	0.0	76.2	178.9	533.4	609.6
BSS ...	0.0	0.0	0.0	457.2	609.6

CROSS-SECTION NUMBER 11
****

XS(I) = 82.890 FSTG(I) = 1424.56

HS ...	1418.5	1420.0	1422.7	1425.8	1427.3
BS ...	0.0	21.3	107.3	121.9	128.0
BSS ...	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 12
****

XS(I) = 95.770 FSTG(I) = 1405.67

HS ...	1402.3	1403.2	1403.8	1406.6	1408.1
BS ...	0.0	74.7	137.1	152.4	158.5
BSS ...	0.0	0.0	0.0	0.0	0.0

1

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	0.080	0.080	0.080	0.080	0.080
REACH 2 ...	0.060	0.060	0.060	0.060	0.060
REACH 3 ...	0.031	0.031	0.031	0.031	0.031
REACH 4 ...	0.034	0.034	0.034	0.034	0.034
REACH 5 ...	0.038	0.038	0.038	0.038	0.038
REACH 6 ...	0.037	0.037	0.037	0.037	0.037
REACH 7 ...	0.034	0.034	0.034	0.034	0.034
REACH 8 ...	0.034	0.034	0.034	0.034	0.034
REACH 9 ...	0.034	0.034	0.034	0.034	0.034
REACH 10 ...	0.036	0.036	0.036	0.036	0.036
REACH 11 ...	0.036	0.036	0.036	0.036	0.036

1

CROSS-SECTIONAL VARIABLES FOR OPT: 1/METRIC
BELOW EXAMPLE-8

PARAMETER	UNITS	VARIABLE
***************************************	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	KM	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	0.810	0.000
2	0.810	-0.900
3	0.810	0.000
4	1.207	0.000
5	1.609	0.100
6	1.609	-0.500
7	1.609	0.000
8	1.609	0.000
9	1.609	0.000
10	1.770	0.000
11	2.253	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPT: 1/METRIC
BELOW EXAMPLE-8

PARAMETER	UNITS	VARIABLE	VALUE
**	*****	********	******
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CMS	QMAXD	2123.4
MAX LATERAL OUTFLOW PRODUCING LOSSES	CMS/M	QLL	-0.034
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	M	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	%	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	M	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

```
     EXAMPLE-9            OPT:5/SUPER

          2          1          0          5          3          0          0          0
1000001100
 0.        5288.5     1.04      5027.      81.        1.43       5027.      0.
5289.0     5288.5     0.        0.         0.         0.         300.       13000.
0.         55.
13000.     50000.     13000.
0.         1.         55.
          5          5          3          4          0          0
          1          3          5
0.         5230.
5220.      5230.      5240.     5250.      5290.
200.       200.       200.      200.       200.
0.
7.9
5123.5     5133.5     5170.     5225.      5290.
200.       350.       600.      725.       800.
0.
8.0
5123.5      5133.5    5170.     5225.      5290.
200.        350.      600.      725.       800.
0.
8.1
5123.5     5133.5     5170.     5225.      5290.
200.       350.       600.      725.       800.
0.
16.0       5037.
5027.      5037.      5100.     5200.      5290.
200.       500.       1000.     1250.      1350.
0.
  .06       .06        .05       .04         .04
  .06       .06        .03       .03         .03
  .06       .06        .03       .03         .03
  .03       .03        .03       .03         .03
   0.5      .1         .1        0.5
  0.
  0.        0.         0.        5288.5
  0.        12.        0.040
          2          3          2          4          0          1
          1          2
 16.0
 5027.      5037.      5057.
 0.         300.       1000.
 0.
 26.
 4027.      4037.      4057.
 0.         300.       1000.
 0.
 0.03       0.03       0.04
 0.5
 0.
```

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE-9
ON
OPT:4/SUPER

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

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***                        ***
***  SUMMARY OF INPUT DATA ***
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INPUT CONTROL PARAMETERS FOR EXAMPLE-9

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	2
TYPE OF RESERVOIR ROUTING	KUI	1
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

IOPUT= 1 0 0 0 0 0 1 1 0 0 0 0
1

EXAMPLE-9 RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
***	*****	********	*******
LENGTH OF RESERVOIR	MILE	RLM	0.00
ELEVATION OF WATER SURFACE	FEET	YO	5288.50
SIDE SLOPE OF BREACH		Z	1.04
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	5027.00
WIDTH OF BASE OF BREACH	FEET	BB	81.00
TIME TO MAXIMUM BREACH SIZE	HOUR	TFH	1.43
ELEVATION (MSL) OF BOTTOM OF DAM	FEET	DATUM	5027.00
VOLUME-SURFACE AREA PARAMETER		VOL	0.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	5289.00
ELEVATION OF TOP OF DAM	FEET	HD	5288.50
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	300.00
DISCHARGE THRU TURBINES	CFS	QT	13000.00

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 55.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO EXAMPLE-9
****

13000.00 50000.00 13000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 55.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF CROSS-SECTIONS	NS	5
MAXIMUM NUMBER OF TOP WIDTHS	NCS	5
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	3
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 3 5

CROSS-SECTIONAL VARIABLES FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 5230.00

HS ...	5220.0	5230.0	5240.0	5250.0	5290.0
BS ...	200.0	200.0	200.0	200.0	200.0
BSS ...	0.0	0.0	0.0	0.0	0.0

```
CROSS-SECTION NUMBER  2
***********************

XS(I) =     7.900   FSTG(I) =    0.00

HS  ...   5123.5  5133.5  5170.0  5225.0  5290.0
BS  ...    200.0   350.0   600.0   725.0   800.0
BSS ...      0.0     0.0     0.0     0.0     0.0


CROSS-SECTION NUMBER  3
***********************

XS(I) =     8.000   FSTG(I) =    0.00

HS  ...   5123.5  5133.5  5170.0  5225.0  5290.0
BS  ...    200.0   350.0   600.0   725.0   800.0
BSS ...      0.0     0.0     0.0     0.0     0.0


CROSS-SECTION NUMBER  4
***********************

XS(I) =     8.100   FSTG(I) =    0.00

HS  ...   5123.5  5133.5  5170.0  5225.0  5290.0
BS  ...    200.0   350.0   600.0   725.0   800.0
BSS ...      0.0     0.0     0.0     0.0     0.0
1


CROSS-SECTION NUMBER  5
***********************

XS(I) =    16.000   FSTG(I) = 5037.00

HS  ...   5027.0  5037.0  5100.0  5200.0  5290.0
BS  ...    200.0   500.0  1000.0  1250.0  1350.0
BSS ...      0.0     0.0     0.0     0.0     0.0


     HS(1, 3) IS GREATER THAN HS(1, 2).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.


     HS(1, 4) IS GREATER THAN HS(1, 3).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE
BASE FLOW IS QUITE SMALL.
1
```

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	0.060	0.060	0.050	0.040	0.040
REACH 2 ...	0.060	0.060	0.030	0.030	0.030
REACH 3 ...	0.060	0.060	0.030	0.030	0.030
REACH 4 ...	0.030	0.030	0.030	0.030	0.030

1

CROSS-SECTIONAL VARIABLES FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	UNITS	VARIABLE
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	0.500	0.000
2	0.100	0.000
3	0.100	0.000
4	0.500	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	5288.50
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

INITIAL CONDITIONS FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	UNITS	VARIABLE	VALUE
CRITICAL DEPTH OF UPSTREAM BOUNDARY	FEET	UPSH	0.00
SLOPE OF DOWNSTREAM CHANNEL	FPM	SOM	12.0000
AVE MANNING'S N FOR DOWNSTREAM CHANNEL		CMN	0.0400

INITIAL CONDITIONS
CROSS-SECTIONAL PARAMETERS FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	VARIABLE	VALUE
NUMBER OF CROSS-SECTIONS	NS	2
MAXIMUM NUMBER OF TOP WIDTHS	NCS	3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	2
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	1
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2

CROSS-SECTIONAL VARIABLES FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	UNITS	VARIABLE
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

```
CROSS-SECTION NUMBER  1
***********************

XS(I) =    16.000   FSTG(I) =    0.00

HS  ...   5027.0  5037.0  5057.0
BS  ...      0.0   300.0  1000.0
BSS ...      0.0     0.0     0.0


CROSS-SECTION NUMBER  2
***********************

XS(I) =    26.000   FSTG(I) =    0.00

HS  ...   4027.0  4037.0  4057.0
BS  ...      0.0   300.0  1000.0
BSS ...      0.0     0.0     0.0
1

            MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
                   (CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
           *******************************************************

                       REACH  1 ... 0.030  0.030  0.040
1

               CROSS-SECTIONAL VARIABLES FOR      OPT:4/SUPER
                              BELOW      EXAMPLE-9

PARAMETER                                            UNITS   VARIABLE
*******************************************          *****   ********

MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN          MILE    DXM(I)
     CROSS-SECTIONS

CONTRACTION - EXPANSION COEFFICIENTS BETWEEN                 FKC(I)
     CROSS-SECTIONS


                    REACH NUMBER        DXM(I)    FKC(I)
                   **************      ********  ********

                         1              0.500     0.000
1
```

DOWNSTREAM FLOW PARAMETERS FOR OPT:4/SUPER
BELOW EXAMPLE-9

PARAMETER	UNITS	VARIABLE	VALUE
***	********	********	******
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

EXAMPLE 10 : OPTION 9--2 DAMS

```
         2         0         1         5         2
1000001100
1936.
5287.     5027.
1.        5288.5    0.        5027.     150.      1.25      5027.     0.
5288.50   5288.50   0.        0.        0.        0.        0.        13000.
  0.        50.
  13000.    13000.
  0.        100.
         3         3         2         1         0         0         0
         1         3
  0.
 5027.     5037.     5112.
  0.       590.      1200.
  0.       0.        0.
  16.
  4817.     4827.    4852.
  0.        1000.    10000.
  0.        0.       20000.
  59.5
  4601.     4606.    4720.
  0.       400.      500.
  0.       0.        0.
  .045     .045      .045
  .045     .045      .045
  .5       .9
  0.       0.
  0.       0.        .000      4701.0
  0.       4601.     100.      1.0
  4702.    4701.5    4701.     0.0       0.        0.        100.      13000.
0.        100.      283.      1118.     3162.
0.        1.        2.        5.        10.
           10.       .045
         3         3         2         4         0         0         0
         1         3
  0.
  4601.     4606.    4660.
   0.      400.      500.
  0.
  20.
  4401.    4406.     4460.
  0.       400.      500.
  0.
  60.
  4000.    4006.     4060.
  0.       400.      500.
  0.
  .045     .045      .045
  .045     .045      .045
  .5       .9
 00.       0.0.      .0        0.
```

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE 10 :
ON
OPTION 9--2 DAMS

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

1

```
******************************
******************************
***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE 10 :

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF DYNAMIC ROUTING REACHES	KKN	2
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	1
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	2
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

IOPUT= 1 0 0 0 0 0 1 1 0 0 0 0

EXAMPLE 10 : RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (ACRES) SA(K)	ELEVATION (FT) HSA(K)
1936.0	5287.00
0.0	5027.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00
0.0	0.00

1

EXAMPLE 10 : RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
LENGTH OF RESERVOIR	MILE	RLM	1.00
ELEVATION OF WATER SURFACE	FEET	YO	5288.50
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	5027.00
WIDTH OF BASE OF BREACH	FEET	BB	150.00
TIME TO MAXIMUM BREACH SIZE	HOUR	TFH	1.25
ELEVATION (MSL) OF BOTTOM OF DAM	FEET	DATUM	5027.00
VOLUME-SURFACE AREA PARAMETER		VOL	0.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	5288.50
ELEVATION OF TOP OF DAM	FEET	HD	5288.50
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	0.00
DISCHARGE THRU TURBINES	CFS	QT	13000.00

CDO SHOULD NOT BE 0.0 IF OVERTOPPING MAY OCCUR

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 50.0000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0

INFLOW HYDROGRAPH TO   EXAMPLE 10 :
****

13000.00   13000.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000   100.0000

1

CROSS-SECTIONAL PARAMETERS FOR OPTION 9--2 DAMS
BELOW   EXAMPLE 10 :

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF CROSS-SECTIONS	NS	3
MAXIMUM NUMBER OF TOP WIDTHS	NCS	3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	2
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1   3

CROSS-SECTIONAL VARIABLES FOR OPTION 9--2 DAMS
BELOW   EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE
***	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	5027.0	5037.0	5112.0
BS ...	0.0	590.0	1200.0
BSS ...	0.0	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 16.000 FSTG(I) = 0.00

HS ...	4817.0	4827.0	4852.0
BS ...	0.0	1000.0	10000.0
BSS ...	0.0	0.0	20000.0

CROSS-SECTION NUMBER 3
****

XS(I) = 59.500 FSTG(I) = 0.00

HS ...	4601.0	4606.0	4720.0
BS ...	0.0	400.0	500.0
BSS ...	0.0	0.0	0.0

1

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	0.045	0.045	0.045
REACH 2 ...	0.045	0.045	0.045

1

CROSS-SECTIONAL VARIABLES FOR OPTION 9--2 DAMS
BELOW EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE
***	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	0.500	0.000
2	0.900	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR OPTION 9--2 DAMS
BELOW EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	4701.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

RESERVOIR AND BREACH PARAMETERS FOR DAM
DOWNSTREAM FROM EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE	VALUE
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	4601.00
WIDTH OF BASE OF BREACH	FEET	BB	100.00
TIME TO MAXIMUM BREACH SIZE	HR	TFH	1.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	4702.00
ELEVATION OF TOP OF DAM	FEET	HD	4701.50
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	4701.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	100.00
DISCHARGE THRU TURBINES	CFS	QT	13000.00

QSPILL(K,1) CFS	HEAD(K,1) FEET
0.	0.0
100.	1.0
283.	2.0
1118.	5.0
3162.	10.0
0.	0.0
0.	0.0
0.	0.0

INITIAL CONDITIONS FOR OPTION 9--2 DAMS
BELOW EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE	VALUE
CRITICAL DEPTH OF UPSTREAM BOUNDARY	FEET	UPSH	0.00
SLOPE OF DOWNSTREAM CHANNEL	FPM	SOM	10.0000
AVE MANNING'S N FOR DOWNSTREAM CHANNEL		CMN	0.0450

```
******************************
******************************
***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE 10 :

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	2
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	1
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	2
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

1

CROSS-SECTIONAL PARAMETERS FOR OPTION 9--2 DAMS
BELOW EXAMPLE 10 :

PARAMETER	VARIABLE	VALUE
**	********	*****
NUMBER OF CROSS-SECTIONS	NS	3
MAXIMUM NUMBER OF TOP WIDTHS	NCS	3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	2
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 3

CROSS-SECTIONAL VARIABLES FOR OPTION 9--2 DAMS
BELOW EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE
***	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	4601.0	4606.0	4660.0
BS ...	0.0	400.0	500.0
BSS ...	0.0	0.0	0.0

```
CROSS-SECTION NUMBER  2
***********************

XS(I) =    20.000   FSTG(I) =    0.00

HS  ...    4401.0  4406.0  4460.0
BS  ...       0.0   400.0   500.0
BSS ...       0.0     0.0     0.0


CROSS-SECTION NUMBER  3
***********************

XS(I) =    60.000   FSTG(I) =    0.00

HS  ...    4000.0  4006.0  4060.0
BS  ...       0.0   400.0   500.0
BSS ...       0.0     0.0     0.0
1

            MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
                   (CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
           *******************************************************

                       REACH  1 ... 0.045  0.045  0.045
                       REACH  2 ... 0.045  0.045  0.045
1

               CROSS-SECTIONAL VARIABLES FOR  OPTION 9--2 DAMS
                             BELOW   EXAMPLE 10 :

PARAMETER                                           UNITS    VARIABLE
********************************************        *****    ********

MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN         MILE     DXM(I)
     CROSS-SECTIONS
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN                 FKC(I)
     CROSS-SECTIONS


                     REACH NUMBER        DXM(I)    FKC(I)
                    **************      ********  ********

                          1               0.500     0.000

                          2               0.900     0.000
1
```

DOWNSTREAM FLOW PARAMETERS FOR OPTION 9--2 DAMS
BELOW EXAMPLE 10 :

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

EXAMPLE 11: MUDFLOW

```
          9          0          0          5          5
1000001100000
 0.0        1.2        0.0                   3          0          1          0
 125.       10.        20.        1.0
 50.        2000.      20000.     50.        50.
 0.0        0.25       0.5        0.75       5.0
          3          2          3          9          0          4
          1          2          3
 0.
 2000.     2050.
 10.       40.
 0.
 0.5
 1000.     1050.
 10.       60.
 0.
 1.0
 0.        50.
 20.       80.
 0.
 0.04      0.05
 0.04      0.05
 0.20      0.20
 0.0       0.0
 0.0        0.0       -40.       0.0
```

PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
EXAMPLE 11:
ON
MUDFLOW

ANALYSIS BY

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT BY
JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND 20910

1

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******************************
******************************
***                        ***
***  SUMMARY OF INPUT DATA ***
***                        ***
******************************
******************************
```

INPUT CONTROL PARAMETERS FOR EXAMPLE 11:

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	9
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	5
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

IOPUT= 1 0 0 0 0 0 1 1 0 0 0 0

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 1.2000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 3
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 1
KSL(LANDSLIDE PARAMETER) = 0

UW= 125.0 VIS= 10.0 SHR= 20.0 POWR= 1.00

INFLOW HYDROGRAPH TO EXAMPLE 11:
****

50.00 2000.00 20000.00 50.00 50.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 0.2500 0.5000 0.7500 5.0000

1

CROSS-SECTIONAL PARAMETERS FOR MUDFLOW
BELOW EXAMPLE 11:

PARAMETER	VARIABLE	VALUE
***	********	*****
NUMBER OF CROSS-SECTIONS	NS	3
MAXIMUM NUMBER OF TOP WIDTHS	NCS	2
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	3
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	9
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	4
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 2 3

CROSS-SECTIONAL VARIABLES FOR MUDFLOW
BELOW EXAMPLE 11:

PARAMETER	UNITS	VARIABLE
**	*****	********
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	2000.0	2050.0
BS ...	10.0	40.0
BSS ...	0.0	0.0

CROSS-SECTION NUMBER 2
****

XS(I) = 0.500 FSTG(I) = 0.00

HS ...	1000.0	1050.0
BS ...	10.0	60.0
BSS ...	0.0	0.0

CROSS-SECTION NUMBER 3
****

XS(I) = 1.000 FSTG(I) = 0.00

HS ...	0.0	50.0
BS ...	20.0	80.0
BSS ...	0.0	0.0

1

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ... 0.040 0.050
REACH 2 ... 0.040 0.050

1

CROSS-SECTIONAL VARIABLES FOR MUDFLOW
BELOW EXAMPLE 11:

PARAMETER	UNITS	VARIABLE
***	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
**************	********	********
1	0.200	0.000
2	0.200	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR MUDFLOW
BELOW EXAMPLE 11:

PARAMETER	UNITS	VARIABLE	VALUE
**	********	********	*****
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	-40.
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

```
    LANDSLIDE
    TEST

         2         1         0         5         3         0         0         0
1010001100
 0.        200.      0.        100.      100.      0.20      100.      0.
 210.      200.      0.        0.        0.        0.        1500.     100.
 0.0       0.5
 100.      100.      100.
 0.0       1.0       5.0
         7         6         6         0         0         0         1
         1         3         4         5         6         7
 0.0
 105.0     155.      180.      205.0     305.0     405.
 0.        100.      150.      200.      400.      600.
 0.0
 3.4
 102.      152.      177.      202.0     302.0     402.
 0.        250.      375.      500.      700.      900.
 0.
 3.5
 102.      152.      177.      202.      302.      402.
 0.        250.      375.      500.      700.      900.
 0.
 3.65
 102.      152.      177.      202.      302.      402.
 0.        250.      375.      500.      700.      900.
 0.
 3.8
 102.      152.      177.      202.      302.      402.
 0.        250.      375.      500.      700.      900.
 0.
 3.9
 102.      152.      177.      202.      302.      402.
 0.        250.      375.      500.      700.      900.
 0.
 5.0
 100.0     150.      175.      200.0     300.0     400.
 0.        250.      375.      500.      700.      900.
 0.
 0.025     0.025     0.025     0.025     0.025     0.025
 0.025     0.025     0.025     0.025     0.025     0.025
 0.025     0.025     0.025     0.025     0.025     0.025
 0.025     0.025     0.025     0.025     0.025     0.025
 0.025     0.025     0.025     0.025     0.025     0.025
 0.025     0.025     0.025     0.025     0.025     0.025
 0.025     0.025     0.025     0.025     0.025     0.025
  0.0       0.0       0.0       0.0       0.0       0.0
 0.        0.        0.0010    200.00
         3         5         4         5         6  0.01      45.       0.50
  50.      100.      50.
```

Appendix D.12.1-2

```
    0.      5.0       0.06
        2          3          2          5
        1          2
1.0
100.      120.      150.
0.        50.       1000.
0.0
2.0
95.       115.      145.
0.        60.       1500.
0.0
0.05      0.06      0.06
0.05
0.0
0.        0.        0.01
```

```
PROGRAM DAMBRK---VERSION--6/20/88

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH
PRODUCED BY THE DAM BREAK OF
LANDSLIDE
ON

ANALYSIS BY
TEST

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

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HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
NOAA, NATIONAL WEATHER SERVICE
SILVER SPRING, MARYLAND  20910
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*******************************
***                         ***
***  SUMMARY OF INPUT DATA  ***
***                         ***
*******************************
*******************************

INPUT CONTROL PARAMETERS FOR LANDSLIDE
```

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	2
TYPE OF RESERVOIR ROUTING	KUI	1
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	5
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

```
IOPUT= 1 0 1 0 0 0 1 1 0 0  0  0
1
```

LANDSLIDE RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
**	*****	********	*******
LENGTH OF RESERVOIR	MILE	RLM	0.00
ELEVATION OF WATER SURFACE	FEET	YO	200.00
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	100.00
WIDTH OF BASE OF BREACH	FEET	BB	100.00
TIME TO MAXIMUM BREACH SIZE	HOUR	TFH	0.20
ELEVATION (MSL) OF BOTTOM OF DAM	FEET	DATUM	100.00
VOLUME-SURFACE AREA PARAMETER		VOL	0.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	210.00
ELEVATION OF TOP OF DAM	FEET	HD	200.00
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	1500.00
DISCHARGE THRU TURBINES	CFS	QT	100.00

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE)= 0.5000 HRS.
BREX(BREACH EXPONENT) = 0.000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 1

INFLOW HYDROGRAPH TO LANDSLIDE
****

100.00 100.00 100.00

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 5.0000

1

CROSS-SECTIONAL PARAMETERS FOR
BELOW LANDSLIDE

PARAMETER	VARIABLE	VALUE
NUMBER OF CROSS-SECTIONS	NS	7
MAXIMUM NUMBER OF TOP WIDTHS	NCS	6
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 3 4 5 6 7

CROSS-SECTIONAL VARIABLES FOR
BELOW LANDSLIDE

PARAMETER	UNITS	VARIABLE
LOCATION OF CROSS-SECTION	MILE	XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION	FEET	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	FEET	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	FEET	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	FEET	BSS(K,I)

NUMBER OF CROSS-SECTION I
NUMBER OF ELEVATION LEVEL K
1

CROSS-SECTION NUMBER 1
****

XS(I) = 0.000 FSTG(I) = 0.00

HS ...	105.0	155.0	180.0	205.0	305.0	405.0
BS ...	0.0	100.0	150.0	200.0	400.0	600.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

Appendix D.12.2-4

CROSS-SECTION NUMBER 2
****

XS(I) = 3.400 FSTG(I) = 0.00

HS ...	102.0	152.0	177.0	202.0	302.0	402.0
BS ...	0.0	250.0	375.0	500.0	700.0	900.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 3
****

XS(I) = 3.500 FSTG(I) = 0.00

HS ...	102.0	152.0	177.0	202.0	302.0	402.0
BS ...	0.0	250.0	375.0	500.0	700.0	900.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 4
****

XS(I) = 3.650 FSTG(I) = 0.00

HS ...	102.0	152.0	177.0	202.0	302.0	402.0
BS ...	0.0	250.0	375.0	500.0	700.0	900.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

1

CROSS-SECTION NUMBER 5
****

XS(I) = 3.800 FSTG(I) = 0.00

HS ...	102.0	152.0	177.0	202.0	302.0	402.0
BS ...	0.0	250.0	375.0	500.0	700.0	900.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 6
****

XS(I) = 3.900 FSTG(I) = 0.00

HS ...	102.0	152.0	177.0	202.0	302.0	402.0
BS ...	0.0	250.0	375.0	500.0	700.0	900.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

CROSS-SECTION NUMBER 7
****

XS(I) = 5.000 FSTG(I) = 0.00

HS ...	100.0	150.0	175.0	200.0	300.0	400.0
BS ...	0.0	250.0	375.0	500.0	700.0	900.0
BSS ...	0.0	0.0	0.0	0.0	0.0	0.0

HS(1, 3) IS GREATER THAN HS(1, 2).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE BASE FLOW IS QUITE SMALL.

HS(1, 4) IS GREATER THAN HS(1, 3).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE BASE FLOW IS QUITE SMALL.

HS(1, 5) IS GREATER THAN HS(1, 4).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE BASE FLOW IS QUITE SMALL.

HS(1, 6) IS GREATER THAN HS(1, 5).
THIS ADVERSE SLOPE MAY CAUSE PROBLEMS LATER IN THE ROUTING COMPUTATIONS IF THE BASE FLOW IS QUITE SMALL.

1

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	0.025	0.025	0.025	0.025	0.025	0.025
REACH 2 ...	0.025	0.025	0.025	0.025	0.025	0.025
REACH 3 ...	0.025	0.025	0.025	0.025	0.025	0.025
REACH 4 ...	0.025	0.025	0.025	0.025	0.025	0.025
REACH 5 ...	0.025	0.025	0.025	0.025	0.025	0.025
REACH 6 ...	0.025	0.025	0.025	0.025	0.025	0.025

1

CROSS-SECTIONAL VARIABLES FOR
BELOW LANDSLIDE

PARAMETER	UNITS	VARIABLE
**	*****	********
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	0.040	0.000
2	0.030	0.000
3	0.030	0.000
4	0.030	0.000
5	0.030	0.000
6	0.040	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR
BELOW LANDSLIDE

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0010
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	200.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.50
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

LSI = 3 LSN = 5 LSL = 4 LSM = 5 LSU = 6 TSL = 0.0100 ALPHA = 45.000 POR = 0.500
THKSL(K), K = 1, NSLI 50.0 100.0 50.0

BELOW LANDSLIDE

PARAMETER	UNITS	VARIABLE	VALUE
CRITICAL DEPTH OF UPSTREAM BOUNDARY	FEET	UPSH	.00
SLOPE OF DOWNSTREAM CHANNEL	FPM	SOM	5.0000
AVE MANNING'S N FOR DOWNSTREAM CHANNEL		CMN	.0600

CROSS-SECTIONAL PARAMETERS FOR
BELOW LANDSLIDE

PARAMETER	VARIABLE	VALUE
NUMBER OF CROSS-SECTIONS	NS	2
MAXIMUM NUMBER OF TOP WIDTHS	NCS	3
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	2
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	5
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

```
              NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
                       (MAX NUMBER OF HYDROGRAPHS = 6)
             **************************************************

                                  1    2


                        CROSS-SECTIONAL VARIABLES FOR
                              BELOW   LANDSLIDE

PARAMETER                                     UNITS   VARIABLE
*******************************************   *****   ********

LOCATION OF CROSS-SECTION                     MILE    XS(I)
ELEVATION(MSL) OF FLOODING AT CROSS-SECTION   FEET    FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH          FEET    HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV          FEET    BS(K,I)
     (ACTIVE FLOW PORTION)
TOP WIDTH CORRESPONDING TO EACH ELEV          FEET    BSS(K,I)
     (OFF-CHANNEL PORTION)

NUMBER OF CROSS-SECTION                               I
NUMBER OF ELEVATION LEVEL                             K
1


CROSS-SECTION NUMBER  1
************************

XS(I) =     1.000   FSTG(I) =    0.00

HS  ...     100.0   120.0   150.0
BS  ...     100.0   300.0   500.0
BSS ...       0.0     0.0     0.0


CROSS-SECTION NUMBER  2
************************

XS(I) =     2.000   FSTG(I) =    0.00

HS  ...      95.0   115.0   145.0
BS  ...       0.0   300.0   500.0
BSS ...       0.0     0.0     0.0
1
```

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ... 0.060 0.060 0.060

1

CROSS-SECTIONAL VARIABLES FOR
BELOW LANDSLIDE

PARAMETER	UNITS	VARIABLE
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	MILE	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	0.020	0.000

1

DOWNSTREAM FLOW PARAMETERS FOR
BELOW LANDSLIDE

PARAMETER	UNITS	VARIABLE	VALUE
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CFS	QMAXD	0.0
MAX LATERAL OUTFLOW PRODUCING LOSSES	CFS/FEET	QLL	0.000
INITIAL SIZE OF TIME STEP	HOUR	DTHM	0.0100
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	FEET	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	FPM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	FEET	EPSY	0.000
TIME AT WHICH DAM STARTS TO FAIL	HOUR	TFI	0.00

APPENDIX E -- Bottom Slope Profile Table

CROSS-SECTION NO.	LOCATION (Mile or KM)	BOTTOM ELEVATION (Feet or M)	REACH NO.	REACH LENGTH (Mile or KM)	SLOPE (FPM or %)
1	0.01	5027.00			
2	5.01	4965.00	1	5.00	12.40
3	8.51	4920.00	2	3.50	12.86
4	16.01	4817.00	3	7.50	13.73
5	22.51	4805.00	4	6.50	1.85
6	27.51	4788.00	5	5.00	3.40
7	32.51	4762.00	6	5.00	5.20
8	37.51	4752.00	7	5.00	2.00
9	41.01	4736.00	8	3.50	4.57
10	43.01	4729.00	9	2.00	3.50
11	51.51	4654.00	10	8.50	8.82
12	59.51	4601.00	11	8.00	6.62

APPENDIX F -- Reservoir Depletion Table

RESERVOIR DEPLETION TABLE

I	K	TTP(I)	Q(I)	H2	YB	D	SUB	VCOR	OUTVOL	BB	COFR	QI(I)	QBRECH	QSPIL
*	*	******	*****	*******	*******	*******	****	****	*******	***	****	******	******	******
1	0	0.000	13003	5288.55	5288.50	5038.81	1.00	1.00	0.0	0.0	3.10	13000.	0.	13003.
2	1	0.029	13227	5288.55	5283.27	5038.88	1.00	1.00	31.0	1.6	3.10	13000.	224.	13003.
3	1	0.057	14256	5288.55	5278.04	5039.21	1.00	1.00	63.5	3.2	3.10	13000.	1254.	13002.
4	1	0.086	16445	5288.54	5272.81	5039.86	1.00	1.00	99.8	4.9	3.10	13000.	3443.	13002.
5	1	0.114	20053	5288.54	5267.58	5040.87	1.00	1.00	142.9	6.5	3.10	13000.	7052.	13001.
6	1	0.143	25297	5288.53	5262.35	5042.20	1.00	1.00	196.5	8.1	3.10	13000.	12297.	13001.

DEFINITION OF VARIABLES IN RESERVOIR DEPLETION TABLE

PARAMETER	UNITS	VARIABLE
**	*****	********
TIME STEP FROM START OF ANALYSIS		I
ITERATIONS NECESSARY TO SOLVE FLOW EQUATIONS		K
ELAPSED TIME FROM START OF ANALYSIS	HRS	TTP(I)
TOTAL OUTFLOW FROM DAM	CFS or CMS	Q(I)
ELEVATION OF WATER SURFACE AT DAM	FT or M	H2
ELEVATION OF BOTTOM OF BREACH	FT or M	YB
EST DEPTH OF FLOW IMMEDIATELY DOWNSTREAM	FT or M	D
SUBMERGENCE COEFFICIENT		SUB
VELOCITY CORRECTION		VCOR
TOTAL VOLUME DISCHARGED FROM TIME OF BREACH	AC-FT or $10^6 M^3$	OUTVOL
BREACH WIDTH	FT or M	BB
RECTANGULAR BREACH DISCHARGE COEFFICIENT		COFR
INFLOW TO RESERVOIR	CFS or CMS	QI(I)
BREACH OUTFLOW	CFS or CMS	QBRECH
SPILLWAY OUTFLOW	CFS or CMS	QSPIL

APPENDIX G -- Initial Condition Table

I= 1	X= 0.010	YN= 5038.90	DEPN= 11.90	YC= 5033.55	DEPC= 6.55	IFR= 0	ITN= 13	ITC= 13
I= 2	X= 0.510	YN= 5032.75	DEPN= 11.95	YC= 5027.37	DEPC= 6.57	IFR= 0	ITN= 13	ITC= 13
I= 3	X= 1.010	YN= 5026.61	DEPN= 2.01	YC= 5021.17	DEPC= 6.58	IFR= 0	ITN= 13	ITC= 13
I= 4	X= 1.510	YN= 5020.49	DEPN= 12.09	YC= 5015.00	DEPC= 6.60	IFR= 0	ITN= 13	ITC= 13
I= 5	X= 2.010	YN= 5014.38	DEPN= 12.18	YC= 5008.81	DEPC= 6.61	IFR= 0	ITN= 13	ITC= 13
.								
.								
.								
I= 70	X= 54.710	YN= 4644.78	DEPN= 11.98	YC= 4640.95	DEPC= 8.15	IFR= 0	ITN= 11	ITC= 11
I= 71	X= 56.310	YN= 4632.93	DEPN= 10.73	YC= 4629.41	DEPC= 7.21	IFR= 0	ITN= 11	ITC= 11
I= 72	X= 57.910	YN= 4621.23	DEPN= 9.63	YC= 4618.03	DEPC= 6.43	IFR= 0	ITN= 10	ITC= 10
I= 73	X= 59.510	YN= 4609.64	DEPN= 8.64	YC= 4606.61	DEPC= 5.61	IFR= 0	ITN= 10	ITC= 10

(IFR(I),I=1,N)

0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0							

IN= 73 YNN= 4609.64 DEP= 8.64

I= 72	X= 57.910	QIL= 13003.	YIL= 4620.98	DEP= 9.38	ITB= 3
I= 71	X= 56.310	QIL= 13003.	YIL= 4632.79	DEP= 10.59	ITB= 3
I= 70	X= 54.710	QIL= 13003.	YIL= 4644.53	DEP= 11.73	ITB= 3
.					
.					
.					
I= 5	X= 2.010	QIL= 13003.	YIL= 5014.41	DEP= 12.21	ITB= 2
I= 4	X= 1.510	QIL= 13003.	YIL= 5020.53	DEP= 12.13	ITB= 3
I= 3	X= 1.010	QIL= 13003.	YIL= 5026.64	DEP= 12.04	ITB= 3
I= 2	X= 0.510	QIL= 13003.	YIL= 5032.77	DEP= 11.97	ITB= 3
I= 1	X= 0.010	QIL= 13003.	YIL= 5038.92	DEP= 11.92	ITB= 3

(QMI(I),I=1,N)

2171784.	2102186.	2034337.	1968205.	1903761.	1840977.	1779823.	1720271.
1662294.	1605863.	1550950.	1497527.	1445567.	1395046.	1345932.	1298204.
1251831.	1206790.	1163055.	1120600.	1079398.	1039428.	1000662.	963077.
926648.	891351.	857163.	824061.	792020.	761018.	731033.	702042.
674022.	630506.	589405.	550628.	514086.	479691.	447357.	417001.
388539.	355993.	326048.	298564.	273400.	250422.	229501.	210510.
193325.	177829.	163908.	151450.	140350.	130505.	121816.	114190.
106515.	100027.	94596.	90690.	87400.	84133.	81559.	79572.
78074.	76979.	76206.	75685.	75281.	75089.	75018.	75001.
75000.							

INITIAL CONDITIONS

(QDI(I),I=1,N)

13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.	13003.	13003.	13003.	13003.	13003.	13003.	13003.
13003.							

(YI(I),I=1,N)

5038.92	5032.77	5026.64	5020.53	5014.41	5008.25	5002.06	4995.89
4989.65	4983.80	4975.93	4969.33	4962.74	4956.10	4949.58	4942.64
4936.99	4927.44	4920.55	4913.66	4906.78	4899.89	4893.02	4886.11
4879.26	4872.31	4865.55	4858.44	4851.96	4844.35	4838.73	4829.49
4827.67	4825.95	4824.23	4822.49	4820.75	4819.00	4817.24	4815.44
4813.59	4810.39	4806.91	4803.31	4799.70	4795.90	4792.09	4788.19
4784.43	4780.24	4776.87	4774.79	4772.68	4770.56	4768.40	4765.58
4760.03	4755.33	4750.89	4746.99	4741.20	4730.76	4720.34	4709.98
4699.59	4689.35	4678.85	4668.75	4656.58	4644.53	4632.79	4620.98
4609.64							

Definitions of Variables in Initial Conditions Output

I --- Cross section counter

X --- Cross section mile or km

YN --- Normal flow water surface elevation (ft or m), for initial flow at t=0

DEPN --- Normal flow depth (ft or m) for initial flow

YC --- Critical flow water surface elevation (ft or m) for initial flow at $t=0$

DEPC --- Critical flow depth (ft or m) for initial flow

IFR --- Froude number indicator 0 indicates $Fr<1$, 1 indicates $Fr\geq 1$

ITN --- Number of iterations to obtain YN via bi-section solution method

ITC --- Number of iterations to obtain YC via bi-section solution method

IFR --- Froude number indicator for each I^{th} cross section

IN --- Number of cross sections at downstream boundary

YNN --- Water surface elevation (ft or m) at downstream boundary for initial flow

DEP --- Depth (ft or m) at downstream boundary for initial flow

I --- Cross section counter

X --- Cross section mile or km

QIL --- Discharge (cfs or cms) at $t=0$ for I^{th} cross section

YIL --- Computed backwater (downwater) water surface elevation (ft or m) at $t=0$ for I^{th} cross section

DEP --- Depth (ft or m) of YIL elevation

QMI(I) --- Expected maximum flow (cfs or cms) during simulation for each I^{th} cross section (used only for computing flow losses via Eq. (92))

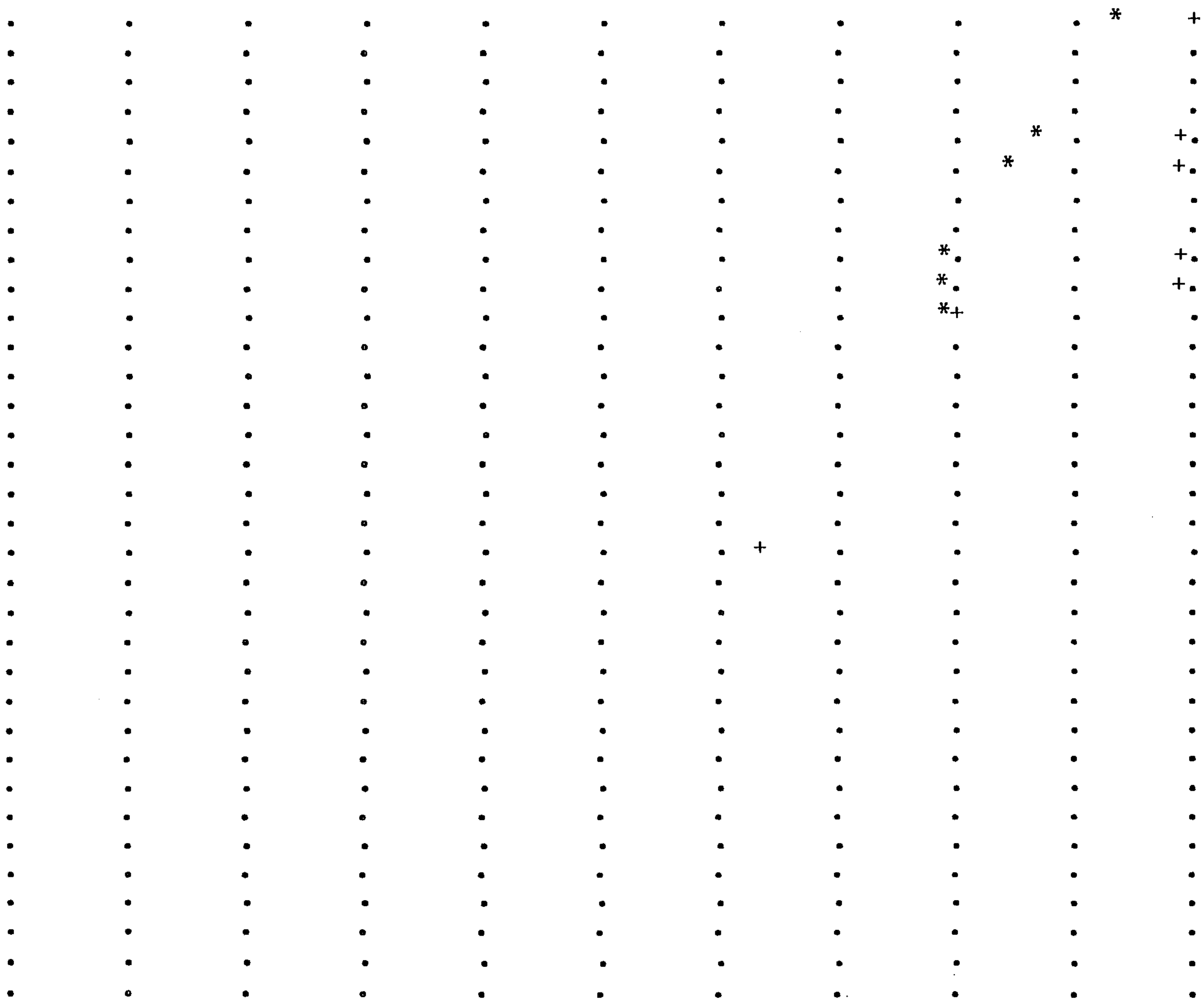
QDI --- Discharge (cfs or cms) at $t=0$ for each I^{th} cross section

YI --- Water surface elevation (ft or m) at $t=0$ for each I^{th} cross section

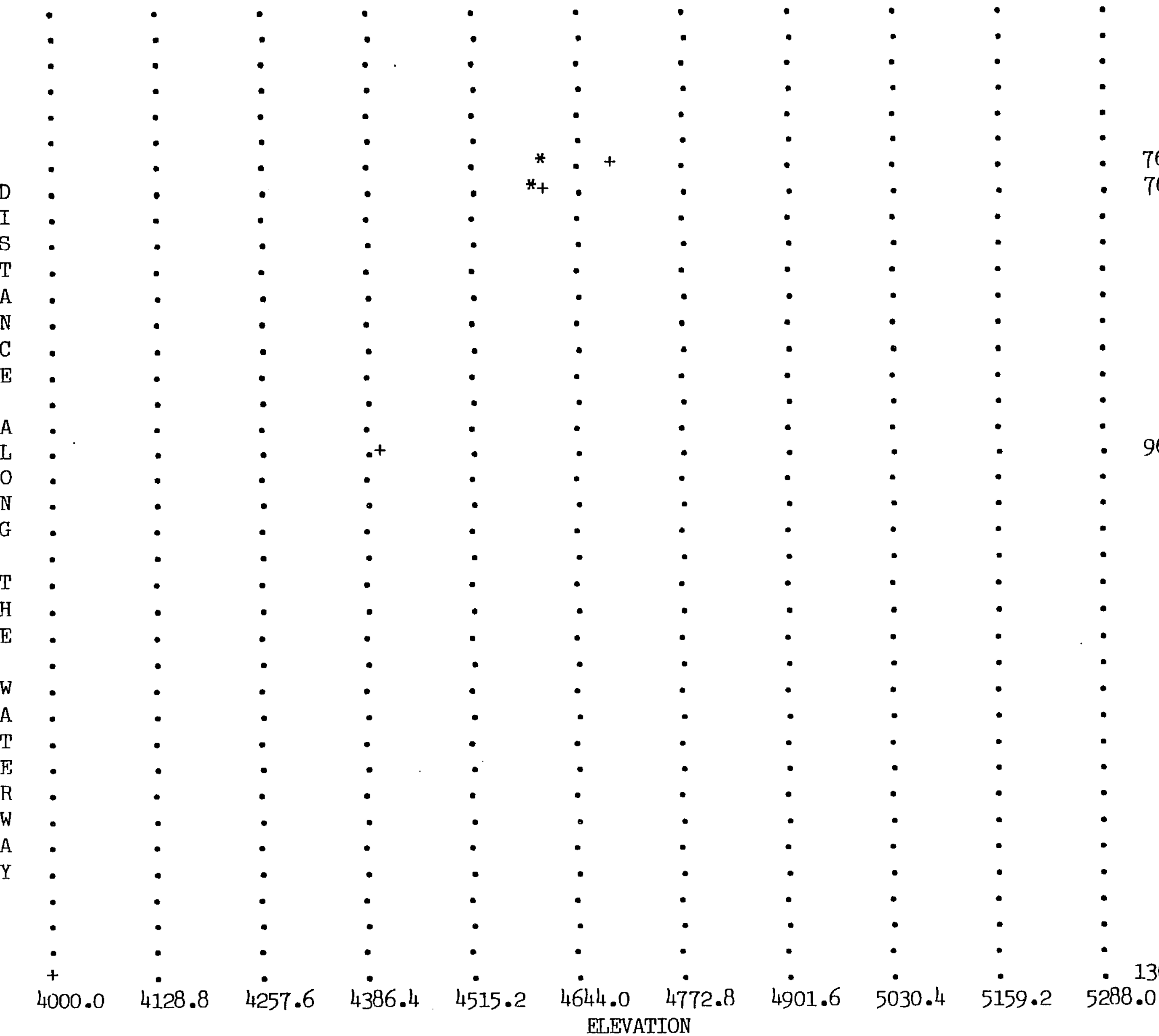
APPENDIX H -- Initial Water Elevations and Bottom Slope Profile Table

PROFILE OF INVERT ELEVATION (HS)
AND INITIAL WATER SURFACE ELEVATION (YI)
ALONG WATERWAY (X)

X(MILES) HS(FEET) — "*" YI(FEET) — "+"



X	HS	YI	DEPTH
0.00	5210.00	5288.00	78.00
7.90	5130.00	5288.00	158.00
11.00	5095.00	5288.00	193.00
16.10	5030.00	5288.00	258.00
16.95	5028.00	5288.00	260.00
17.00	5027.00	5035.70	8.70
33.00	4818.50	4826.12	7.62



76.45	4602.00	4700.00	98.00
76.50	4601.00	4610.00	9.00
96.50	4401.00	4410.00	9.00
136.50	4000.00	4009.50	9.50

APPENDIX I -- Minimal Dynamic Routing Information

TT = 0.0000   DTH = 0.0715   ITERR = 0   QU(1) = 13003.33   YU(1) = 5038.92
QU(N) = 13003.33   YU(N) = 4609.64   FRDM = 0.98   IIFR= 32   FRM = 0.17
IIFM = 51

TT = 0.0000   DTH = 0.0715   ITERR = 1   QU(1) = 13003.33   YU(1) = 5038.92
QU(N) = 13121.15   YU(N) = 4609.64   FRDM = 0.98   IIFR= 32   FRM = 0.17
IIFM = 51

TT = 0.0000   DTH = 0.0715   ITERR = 1   QU(1) = 13003.33   YU(1) = 5038.92
QU(N) = 13222.31   YU(N) = 4609.64   FRDM = 0.98   IIFR = 32   FRM = 0.17
IIFM = 51

TT = 0.0715   DTH = 0.0715   ITERR = 1   QU(1) = 15350.92   YU(1) = 5039.38
QU(N) = 13274.30   YU(N) = 4609.64   FRDM = 0.98   IIFR = 32   FRM = 0.17
IIFM = 51

TT = 0.1430   DTH = 0.0715   ITERR = 2   QU(1) = 25297.23   YU(1) = 5041.29
QU(N) = 13325.02   YU(N) = 4609.64   FRDM = 0.98   IIFR = 32   FRM = 0.17
IIFM = 51

TT = 0.2145   DTH = 0.0715   ITERR = 2   QU(1) = 47003.98   YU(1) = 5044.83
QU(N) = 13342.15   YU(N) = 4609.64   FRDM = 0.98   IIFR = 32   FRM = 0.17
IIFM = 51

Definitions of Variables in Min. Dynamic Routing Table

TT --- Time at which output is given, hrs.
DTH --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1
QU(N) --- Discharge (cfs or cms) at cross section number N (last section at downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM --- Maximum Froude number from cross section number 1 to section number N
IIFR --- Cross section number at which FRDM occurs
FRM --- Minimum Froude number from cross section number 1 to section number N
IIFM --- Cross section number at which FRM occurs

APPENDIX J -- Nonconvergence Information

```
TT = 0.4582   DTH = 0.0016   ITERR = 2   QU(1) = 3164659.99   YU(1) = 5146.79
QU(N) = 53613.91   YU(N) = 4609.65   FRDM = 1.43   IIFR = 59   FRM = 0.09
IIFM = 58


TT = 0.4598   DTH = 0.0016   ITERR = 2   QU(1) = 3157210.99   YU(1) = 5146.81
QU(N) = 55226.17   YU(N) = 4609.65   FRDM = 1.47   IIFR = 59   FRM = 0.09
IIFM = 58
NONCONVERGENCE OCCURRED AT CROSS-SECTION NO.  1  5  7  8  9  10  11  12  13
                TT = 0.460   DTH = 0.002   ITERR = 9

NONCONVERGENCE OCCURRED AT CROSS-SECTION NO.  1  2  3  4  5  6  7  8  9
                TT = 0.460   DTH = 0.001   ITERR = 9


TT = 0.4600   DTH = 0.0002   ITERR = 6   QU(1) = 3156280.99   YU(1) = 5145.99
QU(N) = 58003.66   YU(N) = 4609.65   FRDM = 1.55   IIFR = 59   FRM = 0.07
IIFM = 58
NONCONVERGENCE OCCURRED AT CROSS-SECTION NO.  1  2  3  4  5  6  7  8  9
                TT = 0.460   DTH = 0.001   ITERR = 9


TT = 0.4607   DTH = 0.0007   ITERR = 7   QU(1) = 3153021.99   YU(1) = 5146.66
QU(N) = 62923.57   YU(N) = 4610.32   FRDM = 1.43   IIFR = 59   FRM = 0.08
IIFM = 58
NONCONVERGENCE OCCURRED AT CROSS-SECTION NO.  1  2  3  4  5  6  7  8  9
                TT = 0.461   DTH = 0.001   ITERR = 9


TT = 0.4611   DTH = 0.0003   ITERR = 6   QU(1) = 3151392.99   YU(1) = 5146.43
QU(N) = 63825.36   YU(N) = 4610.32   FRDM = 1.45   IIFR = 59   FRM = 0.07
IIFM = 58
```

Definitions of Variables for Nonconvergence Output (Type J)

```
TT    --- Time at which output is given, hrs.
DTH   --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1
QU(N) --- Discharge (cfs or cms) at cross section number N (last section at
          downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM  --- Maximum Froude number from cross section number 1 to section number N
IIFR  --- Cross section number at which FRDM occurs
FRM   --- Minimum Froude number from cross section number 1 to section number N
```

IIFM --- Cross section number at which FRM occurs

(above are the same as described in Appendix I)

TT --- Time from which time step proceeded when nonconvergence occurred

DTH --- New time step that will be used in next attempt to obtain a converging solution beginning again at time TT and advancing a time step of DTH

ITERR --- Number of iterations (max of a) used when nonconverge occurred

APPENDIX K -- Subcritical/Supercritical Reach Information

TT = 0.0000 DTH = 0.0715 ITERR = 0 QU(1) = 13003.33 YU(1) = 5038.92
QU(N) = 13003.33 YU(N) = 4609.64 FRDM = 0.98 IIFR = 32 FRM = 0.17
IIFM = 51
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
L = 1 KSP = 0 KS1= 1 KSN= 73

TT = 0.0000 DTH = 0.0715 ITERR = 1 QU(1) = 13003.33 YU(1) = 5038.92
QU(N) = 13121.15 YU(N) = 4609.64 FRDM = 0.98 IIFR = 32 FRM = 0.17
IIFM = 51
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
L = 1 KSP = 0 KS1 = 1 KSN = 73

TT = 0.0000 DTH = 0.0715 ITERR = 1 QU(1) = 13003.33 YU(1) = 5038.92
QU(N) = 13222.31 YU(N) = 4609.64 FRDM = 0.98 IIFR = 32 FRM = 0.17
IIFM = 51
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
L = 1 KSP = 0 KS1 = 1 KSN = 73

TT = 0.0715 DTH = 0.0715 ITERR = 1 QU(1) = 15350.92 YU(1) = 5039.38
QU(N) = 13274.30 YU(N) = 4609.64 FRDM = 0.98 IIFR = 32 FRM = 0.17
IIFM = 51
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
L = 1 KSP = 0 KS1 = 1 KSN = 73

TT = 0.1430 DTH = 0.0715 ITERR = 2 QU(1) = 25297.23 YU(1) = 5041.29
QU(N) = 13325.02 YU(N) = 4609.64 FRDM = 0.98 IIFR = 32 FRM = 0.17
IIFM = 51
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
L = 1 KSP = 0 KS1 = 1 KSN = 73

Definitions of Subcritical/Supercritical Reach Information

TT --- Time at which output is given, hrs.
DTH --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1

QU(N) --- Discharge (cfs or cms) at cross section number N (last section at downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM --- Maximum Froude number from cross section number 1 to section number N
IIFR --- Cross section number at which FRDM occurs
FRM --- Minimum Froude number from cross section number 1 to section number N
IIFM --- Cross section number at which FRM occurs

(above are the same as described in Appendix I)

IFR --- Froude number (Fr) indicator (Q = subcritical flow, Fr < 0.95; 1k = supercritical; Fr > 1.05; L = critical, 0.95 < Fr < 1.05)
L --- Number of subcritical/supercritical reach
KSP --- Subcritical/supercritical indicator; 0 = subcritical, 1 = supercritical reach
KS1 --- Cross section number at upstream end of L^{th} subcritical/supercritical reach
KSN --- Cross section number at downstream and of L^{th} subcritical/supercritical reach

APPENDIX M -- Maximum Dynamic Routing Information

TT = 0.0000 DTH = 0.0715 ITERR = 0 QU(1) = 13003.33 YU(1) = 5038.92
QU(N) = 13003.33 YU(N) = 4609.64

I	X(I)	Y	V	A	B	BT	Q	CMM	FKC	WAVHT	DH	FRD	DEPTH
**	******	*******	****	*****	*****	*****	******	******	*****	*****	***	***	*****
1	0.010	5038.92	3.16	4112.	622.	622.	13.003	0.0800	0.00	0.00	6.6	0.2	11.92
5	2.010	5014.41	3.02	4309.	696.	709.	13.003	0.0800	0.00	0.00	6.2	0.2	12.21
9	4.010	4989.65	3.04	4277.	698.	698.	13.003	0.0800	0.00	0.00	6.1	0.2	12.25
13	6.010	4962.74	3.76	3459.	653.	653.	13.003	0.0600	-0.90	0.00	5.3	0.3	10.60
17	8.010	4936.99	3.09	4204.	796.	796.	13.003	0.0600	-0.90	0.00	5.3	0.2	10.57
21	10.010	4906.78	5.84	2225.	603.	603.	13.003	0.0310	0.00	0.00	3.7	0.5	7.38
25	12.010	4879.26	5.77	2255.	615.	615.	13.003	0.0310	0.00	0.00	3.7	0.5	7.33
29	14.010	4851.96	5.38	2417.	645.	645.	13.003	0.0310	0.00	0.00	3.7	0.5	7.49
33	16.010	4827.67	2.57	5051.	1000.	2117.	13.003	0.0340	0.00	0.00	5.1	0.2	10.67
37	19.260	4820.75	2.45	5315.	1150.	3031.	13.003	0.0340	0.00	0.00	4.6	0.2	9.75
41	22.510	4813.59	2.49	5222.	1159.	1159.	13.003	0.0380	0.10	0.00	4.5	0.2	8.59
45	26.510	4799.70	1.86	6992.	2814.	2814.	13.003	0.0380	0.10	0.00	2.5	0.2	8.30
49	30.510	4784.43	1.75	7422.	3378.	6389.	13.003	0.0370	-0.50	0.00	2.2	0.2	12.03
53	34.510	4772.68	1.53	8478.	3641.	9315.	13.003	0.0340	0.00	0.00	2.3	0.2	14.68
57	38.677	4760.03	4.25	3060.	458.	458.	13.003	0.0340	0.00	0.00	6.7	0.3	13.36
61	43.010	4741.20	5.66	2299.	368.	368.	13.003	0.0360	0.00	0.00	6.2	0.4	12.20
65	47.867	4699.59	5.70	2279.	360.	360.	13.003	0.0360	0.00	0.00	6.3	0.4	13.45
69	53.110	4656.58	5.38	2418.	376.	376.	13.003	0.0360	0.00	0.00	6.4	0.4	13.18
73	59.510	4609.64	4.75	2736.	470.	470.	13.003	0.0360	0.00	0.00	5.8	0.3	8.64

FRDM = 0.98 IIFR = 32 FRM = 0.17 IIFM = 51 YU(I) FOR ITERR = 0

YU(I) for ITERR= 0

5038.92	5032.77	5026.64	5020.53	5014.41	5008.25	5002.06	4995.89
4989.65	4983.80	4975.93	4969.34	4962.73	4956.12	4949.54	4942.70
4936.91	4927.49	4920.56	4913.66	4906.78	4899.89	4893.02	4886.11
4879.26	4872.31	4865.55	4858.44	4851.96	4844.35	4838.73	4829.49
4827.67	4825.95	4824.22	4822.49	4820.75	4819.00	4817.24	4815.44
4813.59	4810.39	4806.91	4803.31	4799.70	4795.90	4792.09	4788.19
4784.43	4780.24	4776.87	4774.79	4772.68	4770.56	4768.40	4765.58
4760.02	4755.33	4750.89	4746.99	4741.20	4730.77	4720.34	4709.99
4699.58	4689.37	4678.83	4668.78	4656.54	4644.57	4632.74	4621.04
4609.56							

Appendix M-2

QU(I) FOR ITERR= 0

13003.	13002.	13002.	13000.	13002.	13002.	13000.	13003.
13005.	12999.	13006.	12986.	13020.	12959.	13063.	12910.
13061.	13325.	13029.	13006.	13002.	13004.	13004.	13004.
13004.	13004.	13008.	13004.	13003.	13001.	13000.	12999.
13003.	13002.	13000.	12998.	12999.	12998.	13000.	12995.
13006.	12999.	13013.	12988.	13020.	12972.	13031.	12984.
13020.	12993.	12946.	13037.	12980.	13016.	12997.	13024.
12987.	13006.	13004.	12998.	13005.	12999.	13008.	12993.
13015.	12985.	13028.	12973.	13046.	12943.	13078.	12902.
13130.							

YU(I) FOR ITERR= 1

5038.92	5032.77	5026.64	5020.53	5014.41	5008.25	5002.06	4995.89
4989.65	4983.80	4975.93	4969.34	4962.73	4956.12	4949.54	4942.70
4936.91	4927.49	4920.56	4913.66	4906.78	4899.89	4893.02	4886.11
4879.26	4872.31	4865.55	4858.44	4851.96	4844.35	4838.73	4829.49
4827.67	4825.95	4824.22	4822.49	4820.75	4819.00	4817.24	4815.44
4813.59	4810.39	4806.91	4803.31	4799.70	4795.90	4792.09	4788.19
4784.43	4780.24	4776.87	4774.79	4772.68	4770.56	4768.40	4765.58
4760.02	4755.33	4750.89	4746.99	4741.20	4730.76	4720.34	4709.99
4699.58	4689.37	4678.83	4668.78	4656.54	4644.57	4632.74	4621.04
4609.57							

QU(I) FOR ITERR= 1

13003.	13002.	13002.	13000.	13002.	13002.	13000.	13003.
13005.	12999.	13006.	12986.	13020.	12959.	13063.	12910.
13060.	13323.	13029.	13006.	13002.	13004.	13004.	13004.
13004.	13004.	13008.	13004.	13003.	13001.	13000.	12999.
13003.	13002.	13000.	12998.	12999.	12998.	13000.	12995.
13006.	12999.	13013.	12988.	13020.	12972.	13031.	12984.
13020.	12993.	12946.	13037.	12980.	13017.	12997.	13024.
12987.	13007.	13004.	12998.	13005.	13000.	13007.	12994.
13015.	12986.	13027.	12975.	13043.	12947.	13073.	12909.
13121.							

Q1 = 13003.33   QN = 13121.15   SMI = 13003.33   SMO = 13121.15   SMS =
3826.65   CON = -30.06

Definitions of Variables in Type M Output

TT --- Time at which output is given, hrs.
DTH --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) elevation (ft or m) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1

QU(N) --- Discharge (cfs or cms) at cross section number N (last section at downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM --- Maximum Froude number from cross section number 1 to section number N
IIFR --- Cross section number at which FRDM occurs
FRM --- Minimum Froude number from cross section number 1 to section number N
IIFM --- Cross section number at which FRM occurs

(above are the same as described in Appendix I)

I --- Cross section counter
X(I) --- Cross section distance (miles or km)
Y --- Water surface elevation (ft or m)
V --- Velocity (ft/sec or m/sec)
A --- Wetted active cross-sectional area (ft^2 or m^2)
B --- Wetted active cross-sectional topwidth (ft or m)
BT --- Wetted total (acrive and dead) cross-sectional topwidth (ft or m)
Q --- Discharge (cfs or cms)
CMM --- Manning n
FKC --- Expansion/contraction coefficient
WAVHT --- Wave height (ft); difference between Y and initial (t=0) water surface elevation
DH --- Hydraulic depth (ft) = A/B
FRD --- Froude number
DEPTH --- Depth (ft or m)

(above are the same as described in Appendix L)

YU(I) --- Water surface elevations (ft or m) computed during Newton-Raphson iteration (ITERR)
QU(I) --- Discharge (cfs or cms) computed during Newton-Raphson iteration (ITERR)
Q1 --- Discharge (cfs or cms) at upstream boundary at time TT
QN --- Discharge (cfs or cms) at downstram boundary at time TT
SMI --- Sum of all inflows (cfs or cms) (upstream boundary + lateral flows) up to an including time TT
SMO --- Sum of all outflow (cfs or cms) at downstream boundary up to an including time TT
SMS --- Sum of all $\Delta s/\Delta t$ (cfs or cms) up to and including time TT, in which Δs is the differential storage occurring along the routing reach over the Δt time interval
CON --- Conservation as percentage average max flow, where conservation = inflow - outflow = change in storage/Δt

APPENDIX N -- Type M Output + Sequent Depth Iteration Information

TT = 0.0000 DTH = 0.0500 ITERR = 0 QU(1) = 100.00 YU(1) = 1003.96
QU(N) = 100.00 YU(N) = 879.05

I	X(I)	Y	V	A	B	BT	Q	CMM	FKC	WAVHT	DH	FRD	DEPTH
**	******	*******	*****	*****	****	****	*****	******	****	*****	****	***	*****
1	0.000	1003.96	1.28	78.	40.	40.	0.100	0.0600	0.00	0.00	2.0	0.2	3.96
4	1.500	1002.72	0.19	522.	102.	102.	0.100	0.0600	0.00	0.00	5.1	0.0	10.22
7	3.000	1002.71	0.07	1568.	177.	177.	0.103	0.0600	0.00	0.00	8.9	0.0	17.71
10	4.500	1002.70	0.03	3176.	252.	252.	0.107	0.0600	0.00	0.00	12.6	0.0	25.20
13	5.250	1002.70	0.03	3499.	265.	265.	0.109	0.0600	0.00	0.00	13.2	0.0	26.45
16	6.500	1002.70	0.05	2040.	202.	202.	0.103	0.0600	0.00	0.00	10.1	0.0	20.20
19	8.000	1002.69	0.12	806.	127.	127.	0.100	0.0600	0.00	0.00	6.3	0.0	12.69
22	9.500	1002.44	0.82	122.	49.	49.	0.100	0.0100	0.00	0.00	2.5	0.1	4.94
25	10.250	996.33	11.36	9.	13.	13.	0.100	0.0100	0.00	0.00	0.7	2.5	1.33
28	11.500	971.70	6.95	14.	17.	17.	0.100	0.0100	0.00	0.00	0.8	1.3	1.70
31	13.000	941.53	8.52	12.	15.	15.	0.100	0.0100	0.00	0.00	0.8	1.7	1.53
34	14.500	911.64	7.49	13.	16.	16.	0.101	0.0100	0.00	0.00	0.8	1.5	1.64
37	15.250	902.80	1.22	82.	40.	40.	0.100	0.0600	0.00	0.00	2.0	0.2	4.05
40	16.500	896.55	1.22	82.	40.	40.	0.100	0.0600	0.00	0.00	2.0	0.2	4.05
43	18.000	889.05	1.22	82.	40.	40.	0.100	0.0600	0.00	0.00	2.0	0.2	4.05
46	19.500	881.55	1.22	82.	41.	41.	0.100	0.0600	0.00	0.00	2.0	0.2	4.05

FRDM = 2.46 IIFR = 25 FRM = 0.00 IIFM = 12

IFR = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 2 1 1 1 1 1 1 1 1 1 1 1 0 0 0 0 0 0 0 0 0 0 0

L = 1 KSP = 0 KS1 = 1 KSN = 24
L = 2 KSP = 1 KS1 = 24 KSN = 35
L= 3 KSP = 0 KS1 = 36 KSN = 47

I = 35 YA = 955.96 F = -220470.97 YCT = 16.99
I = 35 YA = 931.43 F = -30748.68 YCT = 8.81
I = 35 YA = 919.17 F = -4711.91 YCT = 4.72
I = 35 YA = 913.04 F = -835.56 YCT = 2.68
I = 35 YA = 909.98 F = -175.93 YCT = 1.66
I = 35 YA = 908.44 F = -41.42 YCT = 1.15
I = 35 YA = 907.68 F = -8.82 YCT = 0.89
I = 35 YA = 907.29 F = -0.13 YCT = 0.76
I = 35 YA = 907.10 F = 2.22 YCT = 0.70
I = 35 YA = 907.20 F = 1.22 YCT = 0.73
I = 35 YA = 907.25 F = 0.59 YCT = 0.75
I = 35 YA = 907.27 F = 0.24 YCT = 0.76
I = 35 YA = 907.28 F = 0.06 YCT = 0.76
I = 35 YA = 907.29 F = -0.04 YCT = 0.76

KJPS = 35 I = 35 YEXT = 905.31 YS = 907.29 ITS = 13

Appendix N-2

```
K = 0    I = 36    YIL = 906.57    QII = 100.    YA = 900.96    F = 3805.2
K = 1    I = 36    YIL = 906.57    QII = 100.    YA = 901.43    F =  812.2
K = 2    I = 36    YIL = 906.57    QII = 100.    YA = 901.67    F = -215.6
K = 3    I = 36    YIL = 906.57    QII = 100.    YA = 901.55    F =  270.2
K = 4    I = 36    YIL = 906.57    QII = 100.    YA = 901.61    F =   20.8
K = 5    I = 36    YIL = 906.57    QII = 100.    YA = 901.64    F =  -98.7
K = 6    I = 36    YIL = 906.57    QII = 100.    YA = 901.63    F =  -39.4
K = 7    I = 36    YIL = 906.57    QII = 100.    YA = 901.62    F =   -9.4

I = 36     YA = 950.96    F = -220509.94    YCT = 16.99
I = 36     YA = 926.44    F =  -30765.35    YCT =  8.81
I = 36     YA = 914.18    F =   -4718.08    YCT =  4.73
I = 36     YA = 908.05    F =    -838.27    YCT =  2.68
I = 36     YA = 904.98    F =    -177.55    YCT =  1.66
I = 36     YA = 903.45    F =     -42.64    YCT =  1.15
I = 36     YA = 902.68    F =      -9.89    YCT =  0.89
I = 36     YA = 902.30    F =      -1.13    YCT =  0.77
I = 36     YA = 902.11    F =       1.27    YCT =  0.70
I = 36     YA = 902.20    F =       0.25    YCT =  0.73
I = 36     YA = 902.25    F =      -0.40    YCT =  0.75
I = 36     YA = 902.23    F =      -0.06    YCT =  0.74
I = 36     YA = 902.22    F =       0.10    YCT =  0.74
I = 36     YA = 902.22    F =       0.02    YCT =  0.74
I = 36     YII = 901.62    YS = 902.22    YSV = 904.05    ITS = 13
YU(I) FOR ITERR = 0

1003.96   1002.92   1002.75   1002.72   1002.71   1002.71   1002.71   1002.71
1002.70   1002.70   1002.70   1002.70   1002.70   1002.70   1002.70   1002.70
1002.70   1002.70   1002.69   1002.68   1002.65   1002.44   1002.39   1001.90
QU(I) FOR ITERR = 0

100.      100.      100.      100.      100.      101.      105.      109.
111.      114.      115.      115.      114.      114.      110.      106.
103.      100.      100.      100.      100.      100.      100.      100.
YU(I) FOR ITERR = 0

1001.90    996.33    991.74    981.46    971.70    961.50    951.67    941.53
931.65    921.55    911.64    906.57
QU(I) FOR ITERR = 0

100.      100.      100.      100.      100.      100.      100.      100.
100.      100.      100.      101.
YU(I) FOR ITERR = 0

904.05    902.80    901.55    899.05    896.55    894.05    891.55    889.05
886.55    884.05    881.55    879.05
QU(I) FOR ITERR = 0

101.      100.      100.      100.      100.      100.      100.      100.
100.      100.      100.      100.
Q1 = 100.00   QN = 100.00   SMI = 200.00   SMO = 200.00   SMS = 23.15   CON = -11.57
```

TT --- Time at which output is given, hrs.
DTH --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1
QU(N) --- Discharge (cfs or cms) at cross section number N (last section at downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM --- Maximum Froude number from cross section number 1 to section number N
IIFR --- Cross section number at which FRDM occurs
FRM --- Minimum Froude number from cross section number 1 to section number N
IIFM --- Cross section number at which FRM occurs

(above are the same as described in Appendix I)

I --- Cross section counter
X(I) --- Cross section distance (miles or km)
Y --- Water surface elevation (ft or m)
V --- Velocity (ft/sec or m/sec)
A --- Wetted active cross-sectional area (ft^2 or m^2)
B --- Wetted active cross-sectional topwidth (ft or m)
BT --- Wetted total (acrive and dead) cross-sectional topwidth (ft or m)
Q --- Discharge (cfs or cms)
CMM --- Manning n
FKC --- Expansion/contraction coefficient
WAVHT --- Wave height (ft); difference between Y and initial (t=0) water surface elevation
DH --- Hydraulic depth (ft or m) = A/B
FRD --- Froude number
DEPTH --- Depth (ft or m)

(above are the same as described in Appendix L)

KSP --- Subcritical/supercritical indicator; 0 = subcritical, 1 = supercritical reach
KS1 --- Cross section number at upstream end of L^{th} subcritical/supercritical reach
KSN --- Cross section number at downstream and of L^{th} subcritical/supercritical reach

(above are the same as described in Appendix K)

I --- Cross section counter
YA --- Sequent water surface elevation (ft or m)
F --- Residual in bi-section method of solving for sequent elevation
YCT --- Z depth used in sequent depth computation (ft or m)
KSPS --- Cross section number where hydraulic jump is located
I --- Cross section counter
YEXT --- Extrapolated water surface elevation (ft or m) from Section I+1 to Section I

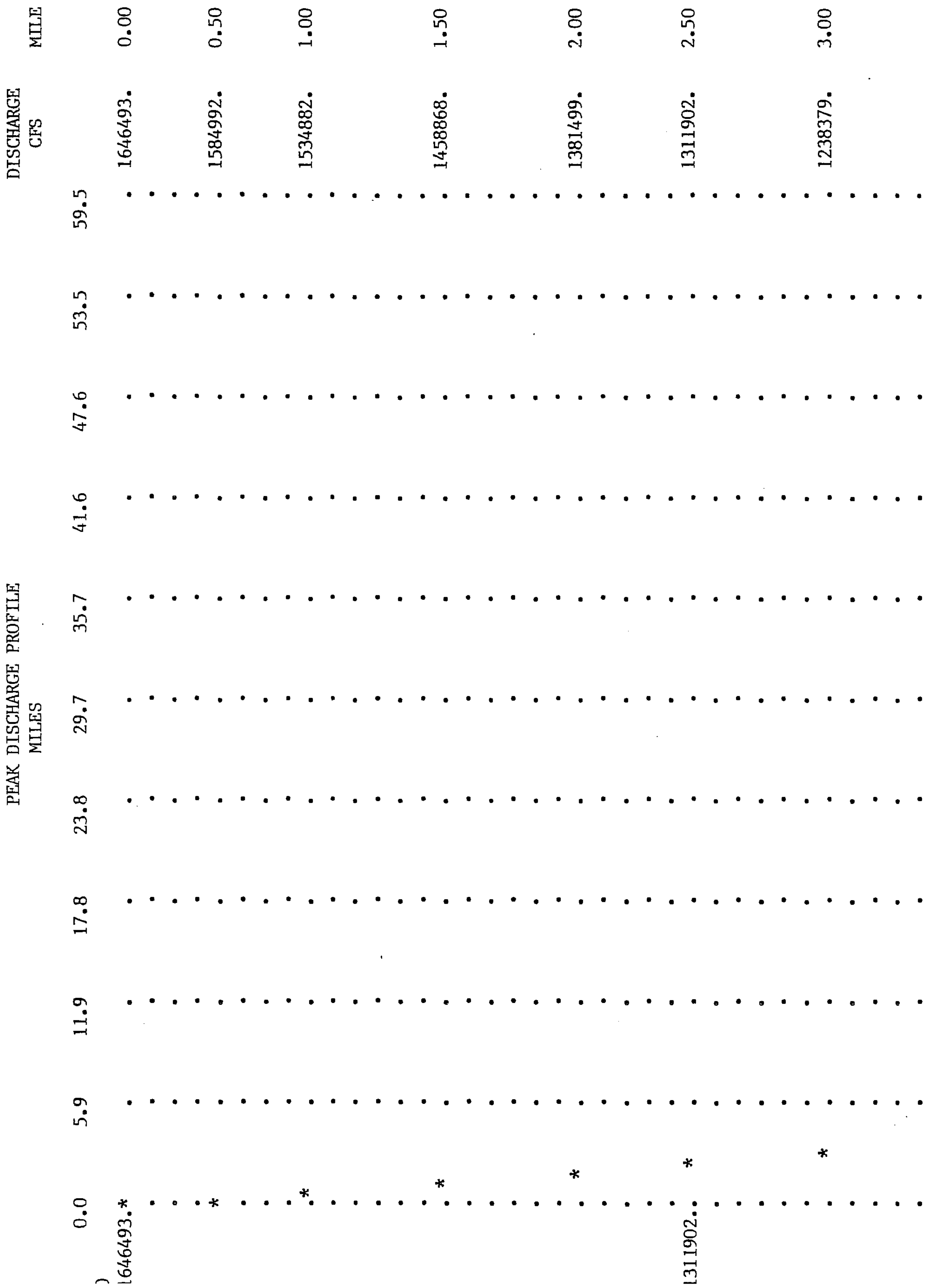
Appendix N-4

YS --- Sequent water surface elevation
ITS --- Number of iterations to computer YS using
K --- Iteration counter
I --- Cross section counter
YIL --- Water surface elevation (ft or m) at section I-1
QIL --- Discharge (cfs or cms) at Section I
YA --- Computed water surface elevation (ft or m) at section I
F --- Residual in bi-section method of solving downwater equation for YA
I --- Cross section counter
YII --- Downwater computed water surface elevation (ft or m)
YS --- Sequent water surface elevation at section I
ITS --- Number of iterations

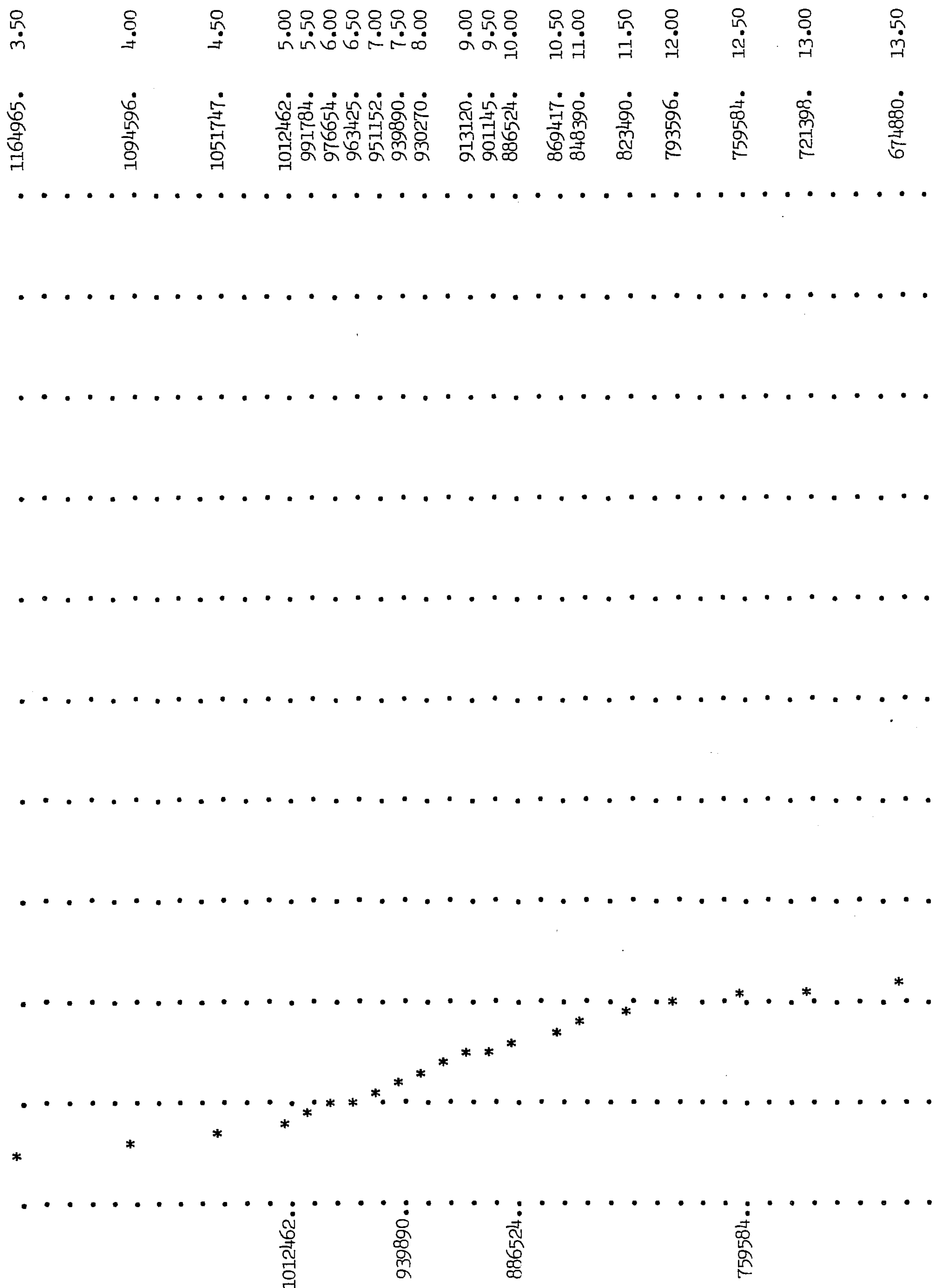
YU(I) --- Water surface elevations (ft or m) computed during Newton-Raphson iteration (ITERR)
QU(I) --- Discharge (cfs or cms) computed during Newton-Raphson iteration (ITERR)
Q1 --- Discharge (cfs or cms) at upstream boundary at time TT
QN --- Discharge (cfs or cms) at downstram boundary at time TT
SMI --- Sum of all inflows (cfs or cms) (upstream boundary + lateral flows) up to an including time TT
SMO --- Sum of all outflow (cfs or cms) at downstream boundary up to an including time TT
SMS --- Sum of all Δs/Δt (cfs or cms) up to and including time TT, in which Δs is the differential storage occurring along the routing reach over the Δt time interval
CON --- Conservation as percentage average max flow, where conservation = inflow - outflow - change in storage/Δt

(above are the same as described in Appendix M)

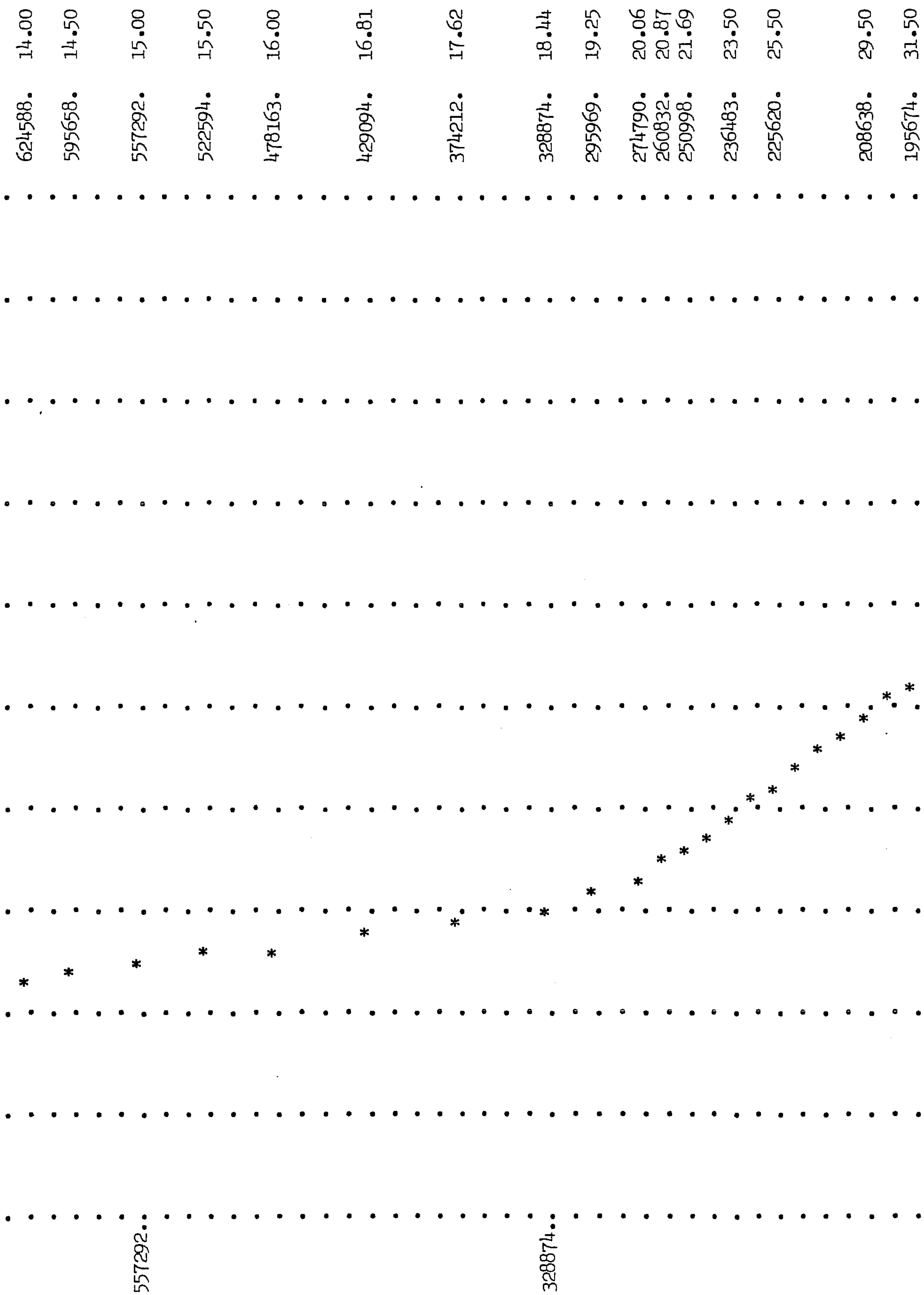
APPENDIX O -- Maximum Discharge Profile Plot



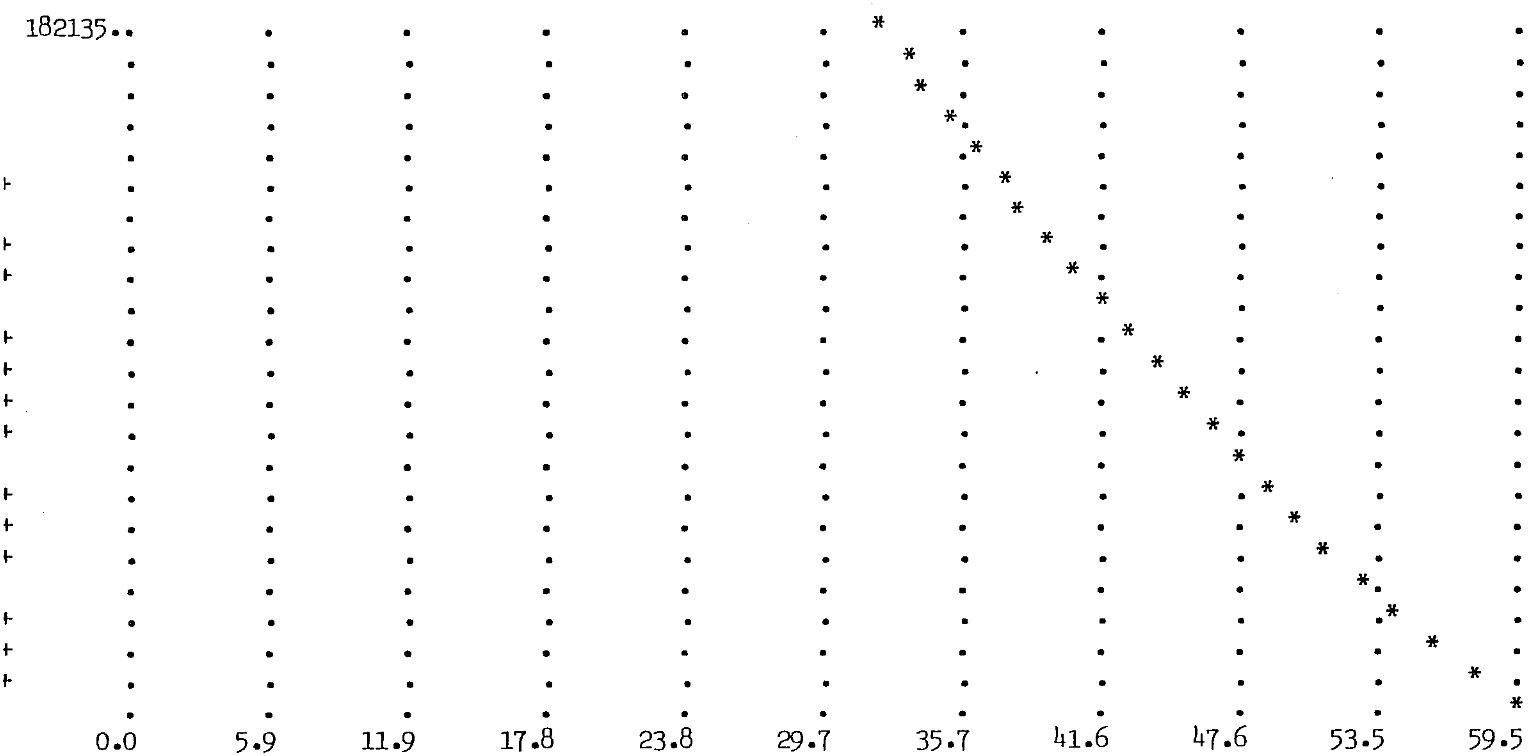
Appendix O-2


3.50 1164965.
4.00 1094596.
4.50 1051747.
5.00 1012462.
5.50 991784.
6.00 976654.
6.50 963425.
7.00 951152.
7.50 939890.
8.00 930270.
9.00 913120.
9.50 901145.
10.00 886524.
10.50 869417.
11.00 848390.
11.50 823490.
12.00 793596.
12.50 759584.
13.00 721398.
13.50 674880.
1012462.
939890.
886524.
759584.

Appendix O-3



14.00
14.50
15.00
15.50
16.00
16.81
17.62
18.44
19.25
20.06
20.87
21.69
23.50
25.50
29.50
31.50
624588.
595658.
557292.
522594.
478163.
429094.
374212.
328874.
295969.
274790.
260832.
250998.
236483.
225620.
208638.
195674.
557292.
328874.



182135.	32.50
165690.	33.50
152092.	34.50
140946.	35.50
132088.	36.50
118509.	38.67
106042.	42.00
91675.	47.86
80130.	53.10
68182.	59.50

APPENDIX P -- Crest Profile Table

Profile of Crests and Times for Teton-Snake River Below Teton Dam

DISTANCE FROM DAM MILE	MAX ELEV FEET	MAX FLOW CFS	TIME MAX ELEV-HRS	MAX VEL FPS	FLOOD ELEV FEET	TIME FLOOD ELEV-HRS
********	********	********	********	*******	**********	**********
0.000	5121.72	1646493	1.375	19.84	5047.00	0.31
0.500	5113.03	1584991	1.437	19.94	5040.80	0.37
1.000	5104.25	1534882	1.500	19.90	5034.60	0.50
1.500	5095.38	1458868	1.625	19.95	5028.40	0.56
2.000	5086.58	1381499	1.687	19.82	5022.20	0.69
2.500	5077.76 *	1311901	1.812	19.59	5016.00	0.75
3.000	5069.04 *	1238379	1.937	19.17	5009.80	0.87
3.500	5060.43 *	1164965	2.062	18.69	5003.60	1.00
4.000	5051.28 *	1094596	2.250	18.01	4997.40	1.06
4.500	5040.84 *	1051746	2.312	17.11	4991.20	1.19
5.000	5023.76	1012462	2.437	22.07	4985.00	1.31
5.500	5014.18	991783	2.500	18.48	4979.43	1.37
6.000	5003.75	976654	2.625	15.91	4973.86	1.50
6.500	4993.28	963425	2.750	13.80	4968.29	1.56
7.000	4982.90	951151	2.875	11.96	4962.71	1.69
7.500	4972.54	939890	3.006	10.41	4957.14	1.81
8.000	4962.73	930270	3.075	8.79	4951.57	1.94
8.500	4950.04	923261	3.212	10.98	4946.00	2.44
9.000	4943.31	913119	3.350	11.22	4938.27	2.44
9.500	4936.57	901144	3.487	11.52	4930.53	2.50
10.000	4929.81	886524	3.625	11.91	4922.80	2.62
10.500	4923.02	869416	3.831	12.28	4915.07	2.69

* Denotes Max Elevation exceeds max topwidth elevation of cross section

Definitions of Variables in Type P Output

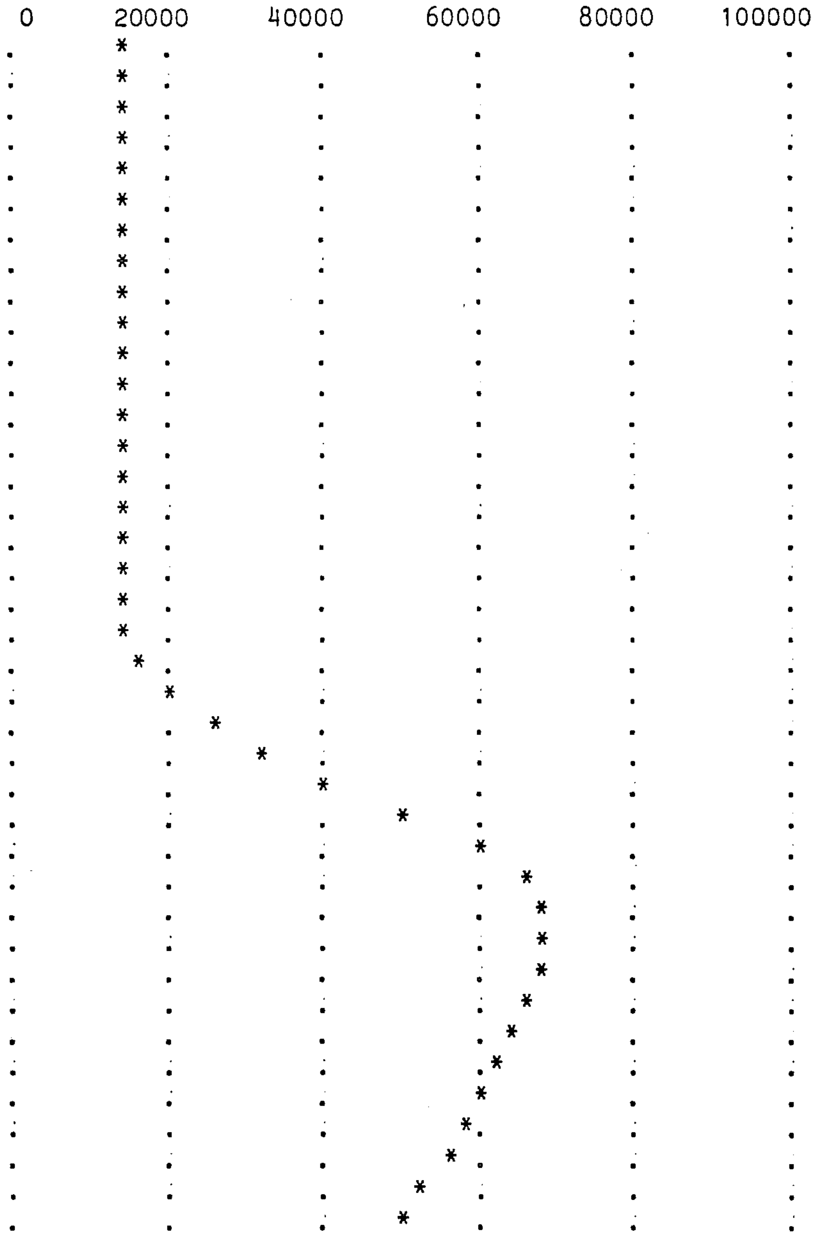
Max Elev	---Maximum computed water surface elevation (ft or m) at the cross section
Max Flow	---Maximum computed discharge (cfs or cms) at the cross section
Time Max	---Time (hr) at which Max Elev occurred
Max Vel	---Maximum computed velocity (ft/sec or m/sec) at the composite cross section
Flood Elev	---Specified elevation (ft or m) for which time of first unundation is desired
Time Flood	---Time (hr) at which Flood Elev is first inundated

APPENDIX Q -- Hydrograph Plot

DISCHARGE HYDROGRAPH FOR TETON-SNAKE RIVER ... STATION NUMBER 73
BELOW TETON DAM AT MILE 59.50

GAGE ZERO = 4601.00 FEET MAX ELEVATION REACHED BY FLOOD WAVE = 4618.91 FEET
FLOOD STAGE = 11.00 FEET
MAX STAGE = 17.91 FEET AT TIME = 33.895 HOURS
MAX FLOW = 68182 CFS AT TIME = 33.895 HOURS

TIME HR	STAGE FEET	FLOW CFS
4	8.6	13385
5	8.6	13385
6	8.6	13299
7	8.6	13298
8	8.6	13298
9	8.6	13298
10	8.6	13298
11	8.6	13298
12	8.6	13298
13	8.6	13298
14	8.6	13298
15	8.6	13264
16	8.6	13262
17	8.6	13262
18	8.6	13262
19	8.6	13262
20	8.6	13264
21	8.6	13283
22	8.6	13281
23	8.6	13577
24	9.2	16019
25	10.3	20950
26	11.5	26454
27	12.5	31832
28	13.8	39305
29	15.4	50217
30	16.7	59618
31	17.5	65159
32	17.8	67624
33	17.9	68178
34	17.8	67547
35	17.7	66180
36	17.4	64368
37	17.2	62276
38	16.9	60029
39	16.5	57692
40	16.2	55320
41	15.9	52952
42	15.5	50629



Appendix Q-2

Definitions of Variables in Hydrograph Plot

```
Gage Zero   --- Bottom elevation (ft or m) of cross section at station number
Stage       --- Maximum elevation (ft or m) of flood wave -- Gage Zero
Max Flow    --- Maximum discharge (cfs or cms)
Flood Stage --- Elevation (ft or m) specified by user -- same as FSTG(I) on
                input data gard group no. 24
```

APPENDIX R -- Computed Water Surface Elevations and Discharges Table

K	TTP(K)	YC(K,I),I=1,NTT					
**	******	***************					
1	0.000	5038.92	4975.93	4927.49	4827.67	4741.20	4609.64
2	0.072	5039.38	4975.93	4927.50	4827.67	4741.20	4609.64
3	0.143	5041.29	4975.93	4927.50	4827.67	4741.20	4609.64
4	0.215	5044.83	4975.93	4927.50	4827.67	4741.20	4609.64
5	0.286	5049.33	4975.93	4927.50	4827.67	4741.20	4609.64
6	0.358	5054.43	4975.93	4927.50	4827.67	4741.20	4609.64
7	0.429	5059.80	4975.93	4927.50	4827.67	4741.20	4609.64
8	0.500	5065.49	4975.93	4927.50	4827.67	4741.20	4609.64
9	0.572	5071.48	4975.93	4927.50	4827.67	4741.20	4609.64
10	0.643	5077.65	4975.93	4927.50	4827.67	4741.20	4609.64
11	0.715	5083.93	4975.93	4927.50	4827.67	4741.20	4609.64
12	0.786	5090.32	4975.94	4927.50	4827.67	4741.20	4609.64
13	0.858	5096.65	4975.94	4927.50	4827.67	4741.20	4609.64

K	TTP(K)	QC(K,I),I=1,NTT					
**	******	***************					
1	0.000	13.00	13.01	13.26	13.00	13.01	13.22
2	0.07	15.35	13.00	13.30	13.00	13.01	13.27
3	0.143	25.30	13.00	13.30	13.00	13.01	13.33
4	0.215	47.00	13.00	13.31	13.00	13.01	13.34
5	0.286	81.83	13.00	13.30	13.00	13.01	13.36
6	0.358	132.34	13.00	13.30	13.00	13.01	13.37
7	0.429	196.60	13.00	13.30	13.00	13.01	13.37
8	0.500	278.35	13.00	13.30	13.00	13.01	13.38
9	0.572	376.54	13.00	13.30	13.00	13.01	13.38
10	0.643	492.01	13.00	13.30	13.00	13.01	13.38
11	0.715	622.42	13.00	13.30	13.00	13.01	13.38
12	0.786	769.90	13.02	13.30	13.00	13.01	13.38
13	0.858	928.60	13.03	13.30	13.00	13.01	13.38

Definitions of Variables in Computed Elevation and Discharge Table

K --- Counter
TTP(K) --- Time (hrs) at which YC( ) and QC( ) occur
YC(K,I) --- Water surface elevation (ft or m) for each time at each station where hydrograph plot is made
QC(K,I) --- Discharge (cfs or cms) for each time at each station where hydrograph plot is made
NTT --- Total number of stations where hydrograph plots are made

APPENDIX S -- Internal Boundary Information

```
TT = 0.0000    DTH = 0.2000    ITERR = 0    QU(1) = 5000.00    YU(1) = 1050.00
QU(N) = 5000.00    YU(N) = 971.31    FRDM = 0.16    IIFR = 25    FRM = 0.00
IIFM = 1

TT = 0.0000    DTH = 0.2000    ITERR = 1    QU(1) = 5000.00    YU(1) = 1049.98
QU(N) = 5000.00    YU(N) = 971.31    FRDM = 0.16    IIFR = 25    FRM = 0.00
IIFM = 1
```

RESERVOIR OUTFLOW INFORMATION

I	K	TT	Q(I)	H2	YB	D	SUB	BB	QU(1)	QBRECH	QOVTOP	QOTHR
1	1	0.000	5000.0	1049.98	1050.00	1011.32	1.00	0.0	5000.0	0.0	0.0	5000.0
24	1	0.000	5000.0	992.09	1005.00	992.04	1.00	0.0	5000.0	0.0	0.0	5000.0

```
TT = 0.0000    DTH = 0.2000    ITERR = 1    QU(1) = 5000.00    YU(1) = 1049.96
QU(N) = 5000.00    YU(N) = 971.31    FRDM = 0.16    IIFR = 25    FRM = 0.00
IIFM = 1
```

RESERVOIR OUTFLOW INFORMATION

I	K	TT	Q(I)	H2	YB	D	SUB	BB	QU(1)	QBRECH	QOVTOP	QOTHR
1	1	0.000	5000.0	1049.96	1050.00	1011.32	1.00	0.0	5000.0	0.0	0.0	5000.0
24	1	0.000	5000.0	992.09	1005.00	992.04	1.00	0.0	5000.0	0.0	0.0	5000.0

Definitions of Variables in Internal Boundary Output

TT --- Time at which output is given, hrs.
DTH --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1
QU(N) --- Discharge (cfs or cms) at cross section number N (last section at downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM --- Maximum Froude number from cross section number 1 to section number N
IIFR --- Cross section number at which FRDM occurs
FRM --- Minimum Froude number from cross section number 1 to section number N
IIFM --- Cross section number at which FRM occurs

(above are the same as described in Appendix I)

I --- Time step counter
K --- Iteration counter (same as ITERR)
TT --- Time (hr)

Appendix S-2

Q(I)	---	Discharge through structure (cfs or cms)
H2	---	Water surface elevation (ft or m) immediately upstream of structure (pool elevation)
YB	---	Elevation (ft or m) of bottom of breach
D	---	Water surface elevation (ft or m) immediately downstream of structure (tailwater elevation)
SUB	---	Submergence correction factor for breach flow
BB	---	Bottom width (ft or m) of breach
QU(1)	---	Discharge (cfs or cms) at upstream end of the reach or pool upstream of the structure
QBREACH	---	Discharge (cfs or cms) through breach
QOVTOP	---	Discharge (cfs or cms) over the top of dam or over crest of bridge embankment
QOTHR	---	Discharge (cfs or cms) of all other flows (Dams: spillways, gates, turbines; Bridge: bridge opening)

APPENDIX T -- Type L Output with Floodplain Compartment Option

TT = 0.0000   DTH = 0.2000   ITERR = 0   QU(1) = 500.00   YU(1) = 101.50
QU(N) = 500.00   YU(N) = 99.00

I	X(I)	Y	V	A	B	BT	Q	CMM	FKC	WAVHT	DISV	FRD	DEPTH
*	*****	******	****	****	****	****	*****	******	****	*****	****	***	*****
1	0.000	101.50	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50
2	0.500	101.00	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50
3	1.000	100.50	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50
4	1.250	100.25	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50
5	1.500	100.00	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50
6	2.000	99.50	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50
7	2.500	99.00	0.67	748.	500.	500.	0.500	0.0400	0.00	0.00	0.0	0.1	1.50

FRDM = 0.10   IIFR = 7   FRM = 0.10   IIFM = 3

K = 1   I = 3   YQU = 100.37   WH = 105.00   SUB = 1.000   QLL = 0.
SQLL = 0.   QQPI = 0.   SQP = 0.   QLUP = 0.   QLDN = 0.

K = 1   I = 4   YQU = 100.12   PEN = 100.00   SUB = 1.000   QLL= 0.
SQLL = 0.   QQPI = 0.   SQP = 0.   QLUP = 0.   QLDN = 0.

K = 2   I = 5   YQU = 99.75   PEN = 99.00   SUB = 1.000   QLL = 0.   SQLL =
0.   QQPI = 0.   SQP = 0.   QLUP = 0.   QLDN = 0.

K = 3   I = 3   YQU = 100.37   WH = 104.50   SUB = 1.000   QLL = 0.   SQLL =
0.   QQPI = 0.   SQP = 0.   QLUP = 0.   QLDN = 0.

K = 3   I = 4   YQU = 100.12   PEN = 100.00   SUB = 1.000   QLL = 0.   SQLL =
0.   QQPI = 0.   SQP = 0.   QLUP = 0.   QLDN = 0.

K = 4   I = 5   YQU = 99.75   PEN = 99.00   SUB = 1.000   QLL = 0.
SQLL = 0.   QQPI = 0.   SQP = 0.   QLUP = 0.   QLDN = 0.

Definition of Variables in Output for Floodplain Compartments (Type L)

TT --- Time at which output is given, hrs.
DTH --- Time step, hrs.
ITERR --- Number of iterations in Newton-Raphson Solution of Saint-Venent Eqs.
QU(1) --- Discharge (cfs or cms) at cross section number 1
YU(1) --- Water surface elevation (ft or m) at cross section number 1
QU(N) --- Discharge (cfs or cms) at cross section number N (last section at downstream boundary)
YU(N) --- Water surface elevation (ft or m) at cross section number N
FRDM --- Maximum Froude number from cross section number 1 to section number N

IIFR --- Cross section number at which FRDM occurs
FRM --- Minimum Froude number from cross section number 1 to section number N
IIFM --- Cross section number at which FRM occurs

(above are the same as described in Appendix I)

I --- Cross section counter
X(I) --- Cross section distance (miles or km)
Y --- Water surface elevation (ft or m)
V --- Velocity (ft/sec or m/sec)
A --- Wetted active cross-sectional area (ft^2 or m^2)
B --- Wetted active cross-sectional topwidth (ft or m)
BT --- Wetted total (active and dead) cross-sectional topwidth (ft or m)
Q --- Discharge (cfs or cms)
CMM --- Manning n
FKC --- Expansion/contraction coefficient
WAVHT --- Wave height (ft); difference between Y and initial (t=0) water surface elevation
DISV --- Cumulative Discharge (A-ft or $10^6 m^3$) released from dam
FRD --- Froude number
DEPTH --- Depth (ft or m)

(above are the same as described in Appendix L)

K --- Floodplain compartment (FPC) counter
I --- Δx subreach counter
YQU --- Average water surface elevation in I^{th} reach
PEN --- New computed water surface elevation in K^{th} FPC
WH --- Elevation of levee crest separating the river and K^{th} FPC
SUB --- Submergence correction factor for flow over levee crest WH
QLL --- Flow over levee crest in I^{th} Δx reach into K^{th} FPC
SQLL --- Total flow over levee crest into K^{th} FPC
QQPI --- Specified hydrograph inflow to K^{th} FPC
SQP --- Total flow discharged by pumps in K^{th} FPC
QLUP --- Flow over levee crest separating K^{th} FPC and next upstream FPC
QLDN --- Flow over levee crest separating K^{th} FPC and next downstream FPC

APPENDIX U -- Crest Profile Table for Conveyance (Floodplain) Option

DISTANCE FROM DAM MILE	MAX ELEV FEET	MAX FLOW CFS	TIME MAX ELEV-HRS	MAX VL FPS	MAX VC FPS	MAX VR FPS	FLOOD ELEV FEET	TIME FLOOD ELEV-HRS
********	*********	********	********	******	******	******	**********	**********
0.000	1050.06 *	109927	0.925	3.28	4.91	3.28	0.00	0.00
0.010	1033.22 *	109927	2.125	6.84	12.39	5.36	0.00	0.00
0.141	1032.49 *	104513	2.175	5.85	10.77	4.61	0.00	0.00
0.273	1031.76 *	101551	2.250	5.55	10.27	4.39	0.00	0.00
0.404	1031.04 *	99309	2.300	5.37	9.99	4.26	0.00	0.00
0.535	1030.32 *	97418	2.375	5.23	9.77	4.16	0.00	0.00
0.667	1029.60 *	95767	2.425	5.11	9.58	4.08	0.00	0.00
0.798	1028.90 *	94283	2.475	5.01	9.42	4.00	0.00	0.00
0.929	1028.20 *	92925	2.550	4.92	9.28	3.94	0.00	0.00
1.061	1027.49 *	91673	2.600	4.89	9.25	3.93	0.00	0.00
1.192	1026.79 *	90513	2.650	4.81	9.14	3.88	0.00	0.00
1.323	1026.09 *	89423	2.725	4.74	9.03	3.83	0.00	0.00
1.454	1025.39 *	88405	2.775	4.70	8.97	3.81	0.00	0.00
1.586	1024.68 *	87443	2.825	4.67	8.93	3.79	0.00	0.00
1.717	1023.97 *	86546	2.875	4.63	8.89	3.77	0.00	0.00
1.848	1023.26 *	85698	2.925	4.58	8.82	3.74	0.00	0.00
1.980	1022.55 *	84903	2.950	4.55	8.78	3.72	0.00	0.00
2.111	1021.83 *	84150	3.000	4.51	8.72	3.70	0.00	0.00
2.242	1021.11 *	83450	3.050	4.48	8.69	3.69	0.00	0.00
2.374	1020.38 *	82795	3.075	4.45	8.65	3.67	0.00	0.00
2.505	1019.65 *	82192	3.125	4.43	8.63	3.66	0.00	0.00
2.636	1018.91 *	81644	3.150	4.41	8.60	3.65	0.00	0.00
2.768	1018.15 *	81153	3.200	4.39	8.58	3.64	0.00	0.00
2.899	1017.39 *	80713	3.225	4.38	8.58	3.64	0.00	0.00
3.030	1016.61 *	80324	3.250	4.38	8.60	3.65	0.00	0.00
3.162	1015.84 *	79979	3.322	4.26	8.39	3.56	0.00	0.00
3.293	1015.08 *	79675	3.322	4.27	8.42	3.57	0.00	0.00
3.424	1014.30 *	79406	3.322	4.28	8.45	3.59	0.00	0.00
3.556	1013.51 *	79170	3.395	4.29	8.49	3.61	0.00	0.00
3.687	1012.70 *	78963	3.395	4.32	8.56	3.64	0.00	0.00
3.818	1011.86 *	78782	3.395	4.35	8.62	3.67	0.00	0.00
3.949	1011.00 *	78624	3.467	4.40	8.71	3.72	0.00	0.00
4.081	1010.10 *	78481	3.467	4.45	8.83	3.77	0.00	0.00
4.212	1009.17 *	78363	3.467	4.51	8.95	3.83	0.00	0.00
4.343	1008.19 *	78248	3.540	4.60	9.11	3.92	0.00	0.00
4.475	1007.15 *	78160	3.540	4.70	9.30	4.01	0.00	0.00
4.606	1006.03 *	78069	3.540	4.84	9.55	4.13	0.00	0.00
4.737	1004.81 *	77998	3.612	5.03	9.91	4.31	0.00	0.00
4.869	1003.43 *	77925	3.612	5.27	10.35	4.52	0.00	0.00
5.000	1002.22 *	77853	3.685	6.35	8.50	4.76	0.00	0.00
5.132	1001.25 *	77791	3.685	6.28	8.48	4.73	0.00	0.00
5.263	1000.29 *	77717	3.685	6.20	8.46	4.69	0.00	0.00
5.395	999.36 *	77658	3.757	6.08	8.36	4.61	0.00	0.00
5.526	998.44 *	77595	3.757	6.02	8.35	4.58	0.00	0.00

5.658	997.54 *	77523	3.830	5.95	8.33	4.54	0.00	0.00
5.789	996.64 *	77469	3.830	5.89	8.32	4.52	0.00	0.00
5.921	995.75 *	77403	3.830	5.84	8.32	4.50	0.00	0.00

* Denotes Max Elevation Exceeds Max Topwidth Elevation at Cross Section

Definition of Variables in Crest Profile Table for Special Conveyance
(Floodplain Option)

Max Elev --- Maximum computed water surface elevation (ft or m) at the cross section
Max Flow --- Maximum computed discharge (cfs or cms) at the cross section
Time Max --- Time (hr) at which Max Elev occurred
Max Vel --- Maximum computed velocity (ft/sec or m/sec) at the composite cross section
Flood Elev --- Specified elevation (ft or m) for which time of first unundation is desired
Time Flood --- Time (hr) at which Flood Elev is first unundated
(above are the same as described in Appendix P)

MAX VL --- Maximum velocity (ft/sec or m/sec) of flow in left floodplain section
MAX VC --- Maximum velocity (ft/sec or m/sec) of flow in channel section
MAX VR --- Maximum velocity (ft/sec or m/sec) of flow in right floodplain section

APPENDIX V -- Type L with Conveyance (Floodplain) Option

TT = 0.0000 DTH = 0.0250 ITERR = 0
QU(1) = 10000.00 YU(1) = 127.97 QU(N) = 10000.00 YU(N) = 77.96

I	X(I)	Y	V	AL	AC	AR	Q	QL	QC	QR	WAVHT	FRD	DEPTH
**	*****	******	****	****	*****	****	*****	****	****	****	*****	***	*****
1	0.000	127.97	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
4	0.750	124.22	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
7	1.500	120.47	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
10	2.250	116.72	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
13	3.000	112.97	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
16	3.750	109.22	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
19	4.500	105.47	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
22	5.250	101.72	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
25	6.000	97.97	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
28	6.750	94.22	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
31	7.500	90.47	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
34	8.250	86.72	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
37	9.000	82.97	4.58	222.	1740.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.97
40	9.750	79.21	4.58	222.	1739.	222.	10.00	0.13	9.61	0.13	0.00	0.3	27.96

FRDM = 0.29 IIFR = 41 FRM = 0.28 IIFM = 1

I --- Cross section counter
X(I) --- Cross section distance (miles or km)
Y --- Water surface elevation (ft or m)
V --- Velocity (ft/sec or m/sec)
AL --- Wetted active cross-sectional area (ft^2 or m^2) in left flood plain
AC --- Wetted active cross-sectional area (ft^2 or m^2) in channel
AR --- Wetted active cross-sectional area (ft^2 or m^2) in right flood plain
Q --- Total discharge (cfs or cms)
QL --- Discharge (cfs or cms) in left flood plain
QC --- Dishcarge (cfs or cms) in channel
QR --- Discharge (cfs or cms) in right floos plain
WAVHT --- Wave height (ft or m); difference between Y and initial (t=0) water surface elevation
FRD --- Froude number
DEPTH --- Depth (ft or m)

APPENDIX W -- Definitions of Subroutines Within DAMBRK

BDARY	Computes coefficients for upstream and downstream boundaries
BRIDGE	Computes coefficients for bridge flow
BWATR	Computes initial steady flow elevations for subcritical flow (backwater solution)
CHANRT	Computes either discharge for a given water surface elevation or vice versa
COFW	Computes broad-crested weir coefficient for a given water surface elevation for floodplain compartments
COMDX	Computes maximum computational distance step for channel ($\Delta x = c\Delta t$)
COMDXR	Computes maximum computational distance step for reservoirs ($\Delta x = \sqrt{gD}\ \Delta t$)
COMPK	Computes total conveyance-elevation table
COMPM	Computes power factor (m) in simplified dam-break analysis
CONV	Computes total conveyance factor K for a given water surface elevation
CONVRT	Converts reservoir volume-elevation to surface area-elevation
CWEIR	Computes broad-crested weir discharge coefficient
DAMBRK	Level pool routing and dam breach for options 1, 2, 3, and 9
DEPTH	Computes flow depth from Manning equation for simplified dambreak analysis to obtain c
DWATR	Computes initial steady flow elevations for supercritical flow (downwater solution)
DXEC	Checks computational distance step against expansion-contraction limitations
DXSLP	Checks computational distance step against bottom slope discontinuity limitations
FRICT	Computes Manning n for a given water surface elevation in the composite channel or in-bank channel

FRICTL	Computes Manning n for a given water surface elevation in the left floodplain section
FRICTR	Computes Manning n for a given water surface elevation in the right floodplain section
GATE	Determines flow through time-dependent gates
HCRIT	Computes critical depth (water surface elevation) for a given flow
HFPC	Computes water surface elevation within floodplain compartment
HNORM	Computes water surface elevation for a given flow assumed to be steady and uniform (normal flow)
HSEQ	Computes sequent depth (water surface elevation) for given flow
IBRDG	Computes initial water surface elevation just upstream of bridge
INITC	Controls the computations for initial conditions of flow and water elevation
INITQ	Computes initial discharges
INTDAM	Computes derivatives and residuals for internal boundaries of dams
INTER	Computes derivatives and residuals for Saint-Venant equations
INTERP	Linear interpolation of discharge hydrograph
INTPXS	Creates cross sections between specified (read-in) sections via linear interpolation
LJUMP	Determines location of hydraulic jump by checking for upstream or downstream movement
MAIN	Main controller for program sequence and reads-in properties for mudflows, upstream and lateral inflow hydrographs, floodplain compartments, time-dependent gates, and landslide
MATRX	Solves matrix for subcritical flow
MATRXC	Solves matrix for supercritical flow
METRI1	Metric conversion for mudflow properties
METRI2	Metric conversion for cross-sectional and dam/bridge properties
METRI3	Metric conversion for boundary conditions, floodplain compartments, time-dependent gates, lateral inflows or outflows

METRI4	Metric conversion for landslide dimension
MIX	Locates subcritical/supercritical flow sub-reaches, boundaries, and types
MKNRT	Muskingum-Cunge routing algorithm for mud flows
OPTAB	Computes floodplain compartment pump discharge for a given head
OUTPUT	Provides tabular output and controls plotted output
PINFLO	Computes broad-crested weir flow over levee between river and floodplain compartment
PLOT	Plots discharge hydrograph
POLFLO	Computes broad-crested weir flow over levee between adjacent floodplain compartments
POTFLO	Computes pump flow out of floodplain compartment
PPLOT	Plots profiles of cross-section invert elevation and initial water surface elevation
PREDIC	Computes routing ratio for peak discharges in simplified dambreak analysis
PRPLOT	Plots profiles of crest elevations, max discharges, times of crest elevations
QDAM	Computes flow through dam for a given water surface elevation
RDAM	Reads-in properties of dams and bridges
RESSEC	Creates reservoir cross sections from surface area-elevation data
ROUTE	Computes routed peak flow in simplified dambreak analysis
RPARM	Computes dimensionless routing parameters in simplified dambreak analysis
RSECT	Reads-in cross section properties and checks for computational distance step size
SDBK	Computes breach outflow for simplified dambreak analysis
SDPRP	Computes spillway flow for simplified dambreak analysis
SDSECT	Computes A, B, DB, CM, DCM for cross sections in simplified dambreak analysis
SECBR	Computes A,B, DB for bridge opening

SECT	Computes cross-sectional properties (B, dB/dh, A, A_o)
SECTF	Computes cross-sectional properties for left and right floodplain sections
SINC	Computes sinuosity coefficient for a given water surface elevation
SLIDE	Determines cross-sectional changes due to landslide
SOLVE	Controls sequence for solving Saint-Venant and external/internal boundary equations
SPILRT	Computes spillway discharge for a given water surface elevation from a routing table
SURFAR	Computes reservoir surface area for a given water surface elevation
TIDE	Computes water surface elevation for downstream boundary for a given time
VTAB	Computes either water surface elevation for a given volume in floodplain compartment or vice versa
WYQMET	Converts discharges or water surface elevations from metric to English or vice versa and prints them
YCTRD	Computes $\bar{z}$ for sequent depth (hydraulic jump) computations
YEND	Computes starting water surface elevation at downstream boundary for initial conditions